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A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

INDEX
TO VOLUME 11

MARCH 1930-FEBRUARY 1931

ENTRIES APPEAR IN THIS INDEX

under names (1) of authors, (2) of research organizations or institutions, and (3) of subjects.

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LIST OF AUTHORS

(For titles of articles, see under names of individual authors in the body of the index)

ANDERSON, ANDREW P.
BALDOCK, R. H.
BARBER, ALVIN B.
BAUMAN, E. W.
BLANEY, H. F.
BUCHANAN, JAMES A.
CRITZ, PAUL F.
CRUM, ROY W.
DAVIS, T. M.
DEAN, A. W.
EDWARDS, R. W.
ELDRIDGE, J. H.
FLEMING, E. M.
FREEMAN, P. J.
HENDERSON, G. H.
HUBBARD, PREVOST
JACKSON, F. H.
JAMES, E. W.
JAMES, N. L.
JOFFE, J. S.
JOHNSON, A. N.
KELLEY, E. F.
MARVIN, CHARLES F., JR.
M'CLINTOCK, DR. MILLER
MOOREFIELD, CHARLES H.
MULLIS, IRA B.
NAGLER, FLOYD A.
OLDER, CLIFFORD
PAULS, J. T.
REAGEL, F. V.
SHEETS, FRANK T.
SOURWINE, J. A.
ST. CLAIR, G. P.
TABER, STEPHEN
TAYLOR, C. A.
TOMS, R. E.
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VOL. 11, NO. 1 ▽ MARCH, 1930

RESEARCH HAS RESULTED IN A BETTER UNDERSTANDING OF STRESSES IN CONCRETE BRIDGE SLABS

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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VOL. 11, NO. 1 MARCH, 1930 R. E. ROYALL, Editor

TABLE OF CONTENTS

Page
Computation of Stresses in Bridge Slabs Due to Wheel Loads - - - - - - 1

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COMPUTATION OF STRESSES IN BRIDGE SLABS DUE TO WHEEL LOADS[a]

By H. M. Westergaard, Professor of Theoretical and Applied Mechanics, University of Illinois, Urbana, Ill.

PART I.—INTRODUCTORY STATEMENT AND DEFINITIONS

SLABS IN HIGHWAY bridges must be designed to support wheel loads in addition to the distributed dead loads. The present investigation is limited to the problem of the stresses contributed by the wheel loads, it being assumed that the influences of the uniform loads may be estimated with sufficient accuracy by available methods.[1] E. F. Kelley[2] published in 1926 a study of the influence of the concentrated loads, in the light of available results of tests, and he proposed formulas for computing the bending moments. The present investigation, which is purely analytical, applies directly to the case of homogeneous elastic slabs. They are subject to accurate analysis by mathematical theory of elasticity. Since the reinforced concrete bridge slab may be assumed to act in certain respects approximately as a homogeneous elastic slab, the results found for the homogeneous elastic slab may be applied in forming a judgment as to the proper formulas for design. It is notable that the results of this analysis do not differ widely from those derived by E. F. Kelley from the tests, in the study referred to.

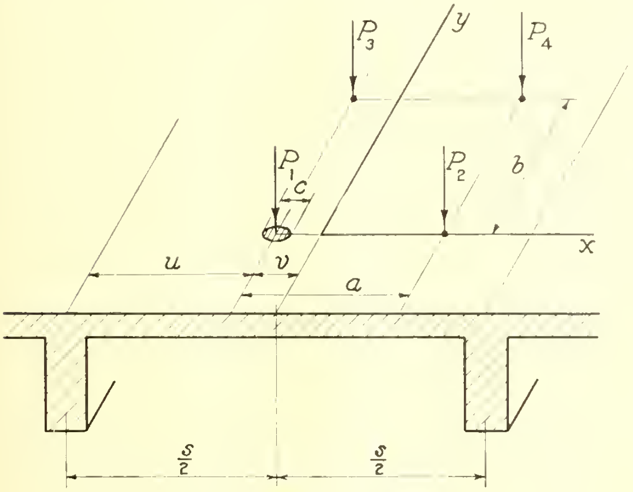

FIGURE 1.—SLAB SUPPORTING WHEEL LOADS

INVESTIGATION OUTLINED

Figure 1 illustrates the problem. The purpose of the analysis is in particular to determine the following effects:

(1) The effect of the load P_1 alone when placed at the center ($v=0$).

(2) The combined effect at the point of application of P_1 produced by the two loads P_1 and P_2 which are separated by the definite distance a, the distance v being chosen so as to produce the greatest possible effect.

(3) The combined effect at the point of application of P_1 produced by the two loads P_1 and P_3, the definite distance b apart, when $v=0$.

(4) The combined effect at the point of application of P_1 produced by the four loads P_1, P_2, P_3, and P_4, which are at the corners of a rectangle with dimensions a and b in the directions of x and y, the distance v being chosen so as to produce the greatest possible effect.

The slab is supported on beams parallel to the direction of y. Most of the computations are based on the assumption that the slab extends sufficiently far in the directions of $+y$ and $-y$ without support by beams in the direction of x to make the influence of edges or beams parallel to the axis of x negligible at the points where the critical stresses exist, thus making it possible for the purpose of analysis to consider the slab to extend infinitely far in the directions of $+y$ and $-y$, without beams or edges in the direction of x. At the same time it will be shown, and illustrated by numerical examples, how the influence of beams in the direction of x may be taken into consideration. When not stated otherwise specifically, the slab will be treated as having simply supported nondeflecting edges along the center lines of the two beams shown in Figure 1. Some computations will be added, however, showing the changes brought about by replacing the simply supported edges by fixed edges. These computations will lead to information about the intermediate cases of partially restrained edges, especially the important case of a continuous slab with several spans in the direction of x.

Each of the four forces P_1, P_2, P_3, and P_4, shown in Figure 1, is the resultant of a wheel pressure which is distributed over a small area. In dealing with the stresses directly under the load P_1, it will be necessary to take into consideration the fact that this load is distributed over an area, but the loads P_2, P_3, and P_4 may be considered as concentrated forces. The load P_1 will be treated as distributed uniformly over a small circle with diameter c. Yet, in expressing effects at some distance from P_1, this load, like the others, may be considered as concentrated at the point of application of the resultant of the pressure.

Two theories of flexure of slabs are used, one of which may be called the ordinary theory, while the other is a special theory. The ordinary theory is based on an assumption which corresponds to the hypothesis of Bernouilli and Navier for beams, that the plane cross section of a beam remains plane and normal to the elastic curve of the beam. The assumption for slabs is that a vertical line drawn through the slab before the bending remains straight and normal to the deflected middle surface after the bending. This assumption applies with satisfactory accuracy to slabs of such proportions as are used commonly in bridges, except for the purpose of expressing the stresses produced by a concentrated load in its immediate vicinity. The difficulty is overcome by use of the special theory in the

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[a] Investigation made for division of tests, U. S. Bureau of Public Roads.

[1] Uniform loads on rectangular slabs, each supported on four sides, may be dealt with, for example, as described in a paper by the writer, entitled "Formulas for the Design of Rectangular Floor Slabs and the Supporting Girders," Proc. American Concrete Institute, vol. 22, 1926, p. 26.

[2] E. F. Kelley, Effective Width of Concrete Bridge Slabs Supporting Concentrated Loads, Public Roads, vol. 7, No. 1, March, 1926, p. 7. This paper contains references to tests and earlier discussions of the same subject.

206328

following manner.[3] The load is introduced as distributed uniformly over the area of a circle with an "equivalent diameter" c' instead of the true diameter c. By use of the special theory, in particular a solution given by A. Nádai,[4] c' is determined so that the ordinary theory, with c' introduced as the diameter of the circle, leads to the same maximum stress at the bottom of the slab directly under the center of the circle, as does the special theory with the true diameter c introduced. The advantage of this procedure is that after introducing c' all the computations may be made according to the ordinary theory, which, naturally, is much simpler than the special theory. Some of the bending moments computed are to be interpreted, accordingly, as equivalent bending moments. They have the significance that the tensile stresses at the bottom of the slab are computed, in the manner applicable in connection with the ordinary theory, by dividing the bending moment per unit of width of the cross section by the section modulus per unit of width; that is, by $\frac{h^2}{6}$, where h is the thickness of the slab.

The study presented here draws extensively on the work of A. Nádai, published first in papers and later in his book on elastic slabs.[5] In a recent investigation of slabs loaded by concentrated forces M. Bergsträsser [6] obtained a satisfactory experimental verification of Nádai's theory.

The results are presented in formulas, tables, and diagrams.

NOTATION

x, y = horizontal rectangular coordinates. The origin of coordinates is at the center of the span as shown in Figure 1, unless specifically stated otherwise. (The y-axis is moved to the left edge in some particular cases.)

r, θ = horizontal polar coordinates.

z = deflection of slab at point x, y.

a, b, u, v = horizontal distances as shown in Figure 1.

h = thickness of the slab.

c = diameter of circle over the area of which the load P_1 is distributed uniformly.

c' = equivalent diameter of the circle over the area of which the load P_1 is to be considered uniformly distributed in order to make the ordinary theory of flexure of the slab lead to the same maximum tensile stress at the bottom of the slab as does the special theory when the diameter is c.

E = modulus of elasticity of the material of the slab.

μ = Poisson's ratio of the material of the slab. In the numerical computations the value assumed is $\mu = 0.15$.

$N = \frac{Eh^3}{12\,(1-\mu^2)}$ = measure of stiffness of the slab.

P, P_1, P_2, P_3, and P_4 = wheel loads.

w = distributed load per unit of area.

p = load per unit of length distributed over a line.

V_x = vertical shear per unit of width of cross section in a section parallel to the y-axis, positive when acting upward on the part having the larger values of x.

V_y = vertical shear per unit of width of cross section in a section parallel to the x-axis, positive when acting upward on the part having the larger values of y.

M_x, M_y = bending moment in the direction of x or y, respectively, per unit of width of cross section, acting upon a section parallel to the y-axis or x-axis, respectively, positive when it produces compression at the top and tension at the bottom.

M_{xy} = twisting moment in the directions of x and y per unit of width of cross section in sections parallel to the axes of x and y, positive when tending to produce compression at the top in the direction of the line $x = y$.

M'_{xy} = value of M_{xy} in particular cases.

R_x = reaction per unit of length at left edge.

Part II.—DERIVATION OF FUNDAMENTAL FORMULAS

FUNDAMENTAL EQUATIONS OF ORDINARY THEORY OF FLEXURE DERIVED

It appears expedient to introduce the analysis by showing briefly the derivations of the general fundamental equations of the ordinary theory of flexure of slabs.[7]

Figure 2 shows three fundamental types of deformation of an element of the slab. They are produced by the bending moments and twisting moments acting on the element. One may visualize the deformation of the element in the general case by imagining the three types existing in the same element at the same time, superimposed one on another.

Figure 3 shows the total forces and couples acting on a small block of the slab extending through the thickness of the slab. In passing from the face with coordinate x to that with coordinate $x+dx$, the bending moment per unit of width, M_x, increases at the rate of $\frac{\partial M_x}{\partial x}$ by the amount $\frac{\partial M_x}{\partial x}\,dx$. The values per unit of width, therefore, may be stated as follows: M_x at the face with the coordinate x; and $M_x + \frac{\partial M_x}{\partial x}\,dx$ at the face with the coordinate $x+dx$. The total moments on the width dy, consequently, may be stated as shown in Figure 3: $M_x dy$ at the face with coordinate x; and $(M_x + \frac{\partial M_x}{\partial x}\,dx)dy$ at the face with coordinate $x+dx$. Similar explanations apply to the bending moment M_y, the twisting moments M_{xy} and M_{yx}, and the shears V_x and V_y.

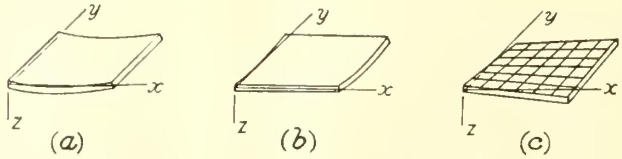

FIGURE 2.—DEFORMATIONS OF ELEMENT OF SLAB, (a) BENDING IN DIRECTION OF x, (b) BENDING IN DIRECTION OF y, (c) TWISTING IN DIRECTIONS OF x AND y

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[3] Described in a previous paper by the writer, Stresses in Concrete Pavements Computed by Theoretical Analysis, Public Roads, vol. 7, No. 2, April, 1926, p. 25, especially pp. 27, 31, and 32.

[4] A. Nádai, Die Biegungsbeanspruchung von Platten durch Einzelkräfte, Schweizerische Bauzeitung, vol. 76, 1920, p. 257; and his book, Die elastischen Platten, 1925 p. 308.

[5] A. Nádai, Die elastischen Platten, Berlin (Julius Springer), 1925.

[6] M. Bergsträsser, Versuche mit freiaufliegenden rechteckigen Platten unter Einzelkraftbelastung, Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, No. 302, 1928.

[7] These derivations may be found at a number of places in the technical literature. See, for example, A. Nádai, Die elastischen Platten, 1925, p. 20; or the paper by W. A. Slater and the writer, Moments and Stresses in Slabs, Proceedings, American Concrete Institute, vol. 17, 1921, p. 415 (or, National Research Council, Reprint and Circular Series, No. 32).

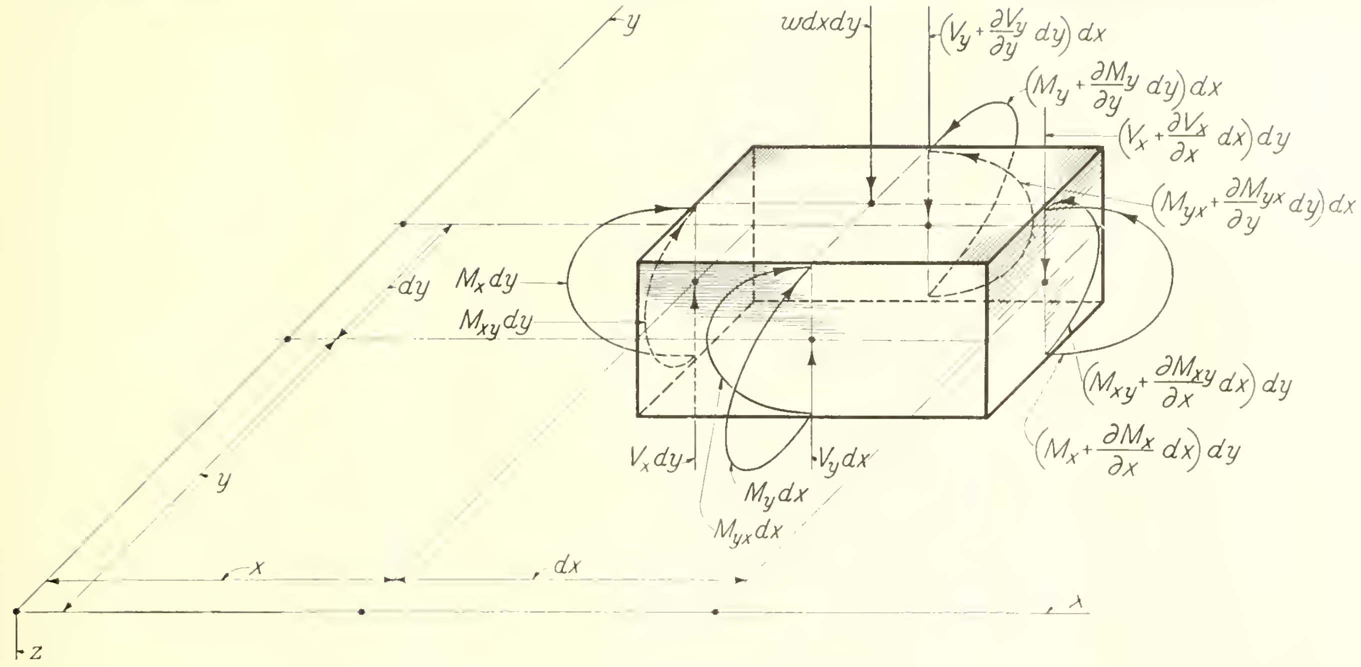
y
$w\,dx\,dy$
$\left(V_y+\frac{\partial V_y}{\partial y}dy\right)dx$
$\left(M_y+\frac{\partial M_y}{\partial y}dy\right)dx$
$\left(V_x+\frac{\partial V_x}{\partial x}dx\right)dy$
$\left(M_{yx}+\frac{\partial M_{yx}}{\partial y}dy\right)dx$
dy
$M_x\,dy$
$M_{xy}\,dy$
$\left(M_{xy}+\frac{\partial M_{xy}}{\partial x}dx\right)dy$
$\left(M_x+\frac{\partial M_x}{\partial x}dx\right)dy$
$V_x\,dy$
$V_y\,dx$
$M_y\,dx$
$M_{yx}\,dx$
y
x
dx
x
z

Figure 3.—Forces and Couples Acting on Element of Slab

One may write three independent equations of equilibrium of the forces and couples. By equating to zero the sum of the vertical forces, and dividing by $dx\,dy$, one finds

$$\frac{\partial V_x}{\partial x}+\frac{\partial V_y}{\partial y}+w=0 \quad \text{(1)}$$

By equating to zero the sum of the moments with respect to an axis through the center of the block, parallel to the y-axis, by discarding the term which is infinitesimal of the third order, $\frac{\partial V_x}{\partial x}\,dx\,dy.\ \tfrac{1}{2}dx$, and again dividing by $dx\,dy$ one finds the equation:

$$\frac{\partial M_x}{\partial x}+\frac{\partial M_{yx}}{\partial y}=V_x \quad \text{(2)}$$

The third equation of equilibrium is similar to equation 2, and is obtained by exchanging the symbols x and y in equation 2:

$$\frac{\partial M_y}{\partial y}+\frac{\partial M_{xy}}{\partial x}=V_y \quad \text{(3)}$$

The twisting moments are moments of horizontal shearing stresses in the vertical sections. By applying the law of equality of shearing stresses in perpendicular sections, one finds

$$M_{xy}=M_{yx} \quad \text{(4)}$$

By substituting the expressions for V_x and V_y, as given in equations 2 and 3, in equation 1, one finds the additional equation of equilibrium,

$$\frac{\partial^2 M_x}{\partial x^2}+2\frac{\partial^2 M_{xy}}{\partial x\partial y}+\frac{\partial^2 M_y}{\partial y^2}=-w \quad \text{(5)}$$

When each straight line drawn vertically through the homogeneous slab before bending remains straight after the bending, the horizontal normal stresses and shearing stresses in the vertical sections will be distributed through the thickness of the slab according to straight-line diagrams, with extreme, equal and opposite values at the top and the bottom and the value zero at the middle. The middle surface of the slab, therefore, is a neutral surface. At the bottom, the tensile stresses σ_x in the direction of x, and σ_y in the direction of y, and the shearing stress τ_{xy} in the directions of x and y are determined as in the case of beams, by dividing the moments by the section modulus, which is $\frac{h^2}{6}$ per unit of width of the section; that is,

$$\sigma_x=\frac{6\,M_x}{h^2},\ \sigma_y=\frac{6\,M_y}{h^2},\ \tau_{xy}=\frac{6\,M_{xy}}{h^2} \quad \text{(6)}$$

The next step is to express the relations between the moments and the deformations. The deformations in Figure 2, (a) and (b), are measured by the curvatures, $-\frac{\partial^2 z}{\partial x^2}$ in the direction of x, and $-\frac{\partial^2 z}{\partial y^2}$ in the direction of y, respectively. The deformation in Figure 2 (c) is measured by the twist, $-\frac{\partial^2 z}{\partial x\partial y}$. The bending moment M_x alone, without the action of M_y and M_{xy}, produces a curvature, $-\frac{\partial^2 z}{\partial x^2}$, in the direction of x, which is expressed as in the case of a beam with rectangular cross section of depth h and width equal to one unit, that is, $-\frac{\partial^2 z}{\partial x^2}=\frac{12 M_x}{Eh^3}$. On account of Poisson's ratio, μ, of lateral contraction to longitudinal extension, this curvature will be accompanied by a curvature in the direction of y, equal to $-\mu$ times the curvature in the direction of x. The bending moment M_x produces no twist of the type $-\frac{\partial^2 z}{\partial x\partial y}$. By expressing in a similar manner the effects of the bending moment M_y, one finds the combined effect of the two bending moments,

$$-\frac{\partial^2 z}{\partial x^2}=\frac{12}{Eh^3}\left(M_x-\mu M_y\right) \quad \text{(7)}$$

$$-\frac{\partial^2 z}{\partial y^2}=\frac{12}{Eh^3}\left(M_y-\mu M_x\right) \quad \text{(8)}$$

The effects of the twisting moments M_{xy} may be found by introducing a new system of horizontal rectangular coordinates, x_1, y_1, with the angle $(xx_1)=45°$, so that $x_1=\frac{1}{\sqrt{2}}(x+y)$, $y_1=\frac{1}{\sqrt{2}}(-x+y)$. When f is any function, one finds

$$\frac{\partial f}{\partial x}=\frac{\partial f}{\partial x_1}\frac{\partial x_1}{\partial x}+\frac{\partial f}{\partial y_1}\frac{\partial y_1}{\partial x}=\frac{1}{\sqrt{2}}\left(\frac{\partial f}{\partial x_1}-\frac{\partial f}{\partial y_1}\right).$$

This result may be written as a statement concerning the operator $\frac{\partial}{\partial x}$: Namely $\frac{\partial}{\partial x}=\frac{1}{\sqrt{2}}\left(\frac{\partial}{\partial x_1}-\frac{\partial}{\partial y_1}\right)$.

One finds in the same manner, $\frac{\partial}{\partial y}=\frac{1}{\sqrt{2}}\left(\frac{\partial}{\partial x_1}+\frac{\partial}{\partial y_1}\right)$, and accordingly, by combining the differential operations:

$$\frac{\partial^2 z}{\partial x\partial y}=\frac{1}{2}\left(\frac{\partial}{\partial x_1}-\frac{\partial}{\partial y_1}\right)\left(\frac{\partial z}{\partial x_1}+\frac{\partial z}{\partial y_1}\right)=\frac{1}{2}\left(\frac{\partial^2 z}{\partial x_1{}^2}-\frac{\partial^2 z}{\partial y_1{}^2}\right).$$

The state of moments, $M_x=0$, $M_y=0$, $M_{xy}\neq 0$, is equivalent to the following state of moments in the directions of x_1 and y_1: $M_{x_1}=M_{xy}$, $M_{y_1}=-M_{xy}$, $M_{x_1y_1}=0$. By using these values in equations 7 and 8, with x and y replaced by x_1 and y_1, and then substituting in the expression for $\frac{\partial^2 z}{\partial x\partial y}$, one finds

$$-\frac{\partial^2 z}{\partial x\partial y}=\frac{12(1+\mu)}{Eh^3}M_{xy} \qquad (9)$$

By this method one finds, furthermore, for the same state of moments,

$$\frac{\partial^2 z}{\partial x^2}=\frac{1}{2}\left(\frac{\partial}{\partial x_1}-\frac{\partial}{\partial y_1}\right)^2 z=\frac{1}{2}\left(\frac{\partial^2 z}{\partial x_1{}^2}-2\frac{\partial^2 z}{\partial x_1\partial y_1}+\frac{\partial^2 z}{\partial y_1{}^2}\right)=0,$$

and likewise $\frac{\partial^2 z}{\partial y^2}=0$. That is, the twisting moment M_{xy} does not contribute to the curvatures, $-\frac{\partial^2 z}{\partial x^2}$ and $-\frac{\partial^2 z}{\partial y^2}$.

The three equations, 7, 8, and 9, express therefore the combined effect of the state of bending moments, M_x and M_y, and twisting moments, M_{xy}.

It is expedient to introduce the following quantity, which is a measure of the stiffness of the slab:

$$N=\frac{Eh^3}{12(1-\mu^2)} \qquad (10)$$

Using this quantity, one finds, by solving equations 7, 8, and 9 for the moments,

$$M_x=N\left(-\frac{\partial^2 z}{\partial x^2}-\mu\frac{\partial^2 z}{\partial y^2}\right) \qquad (11)$$

$$M_y=N\left(-\frac{\partial^2 z}{\partial y^2}-\mu\frac{\partial^2 z}{\partial x^2}\right) \qquad (12)$$

$$M_{xy}=-N(1-\mu)\frac{\partial^2 z}{\partial x\partial y} \qquad (13)$$

By substituting these expressions in equation 5, one obtains the equation of flexure of the slab, stated by Lagrange in 1811, and frequently named after him,

$$\frac{\partial^4 z}{\partial x^4}+2\frac{\partial^4 z}{\partial x^2\partial y^2}+\frac{\partial^4 z}{\partial y^4}=\frac{w}{N} \qquad (14)$$

By introducing the differential operator, known as Laplace's operator for two variables,

$$\Delta=\frac{\partial^2}{\partial x^2}+\frac{\partial^2}{\partial y^2} \qquad (15)$$

Lagrange's equation is restated in the simple form,

$$N\Delta^2 z=w \qquad (16)$$

The vertical shears are expressed in terms of the deflections by substituting the expressions in equations 11, 12, and 13, in equations 2 and 3. One finds

$$V_x=-N\frac{\Delta z}{\partial x}, V_y=-N\frac{\Delta z}{\partial y} \qquad (17)$$

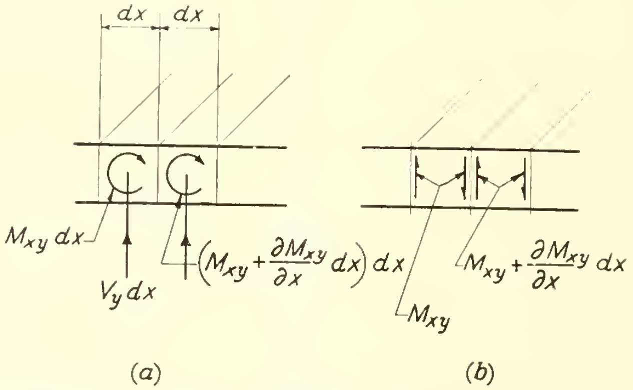
dx
dx
$M_{xy}\,dx$
$V_y\,dx$
$\left(M_{xy}+\frac{\partial M_{xy}}{\partial x}dx\right)dx$
$M_{xy}+\frac{\partial M_{xy}}{\partial x}dx$
M_{xy}
(a)
(b)

FIGURE 4.—TWISTING MOMENTS AND SHEARS AT EDGE

Figure 4 shows an edge of the slab. The twisting couples in Figure 4 (a) are resultants of horizontal shearing forces. These couples are equivalent to the pairs of vertical forces shown in Figure 4 (b). The two vertical forces at the boundary between the two blocks leave a surplus upward force equal to $\frac{\partial M_{xy}}{\partial x}dx$, that is, $\frac{\partial M_{xy}}{\partial x}$ per unit of length. This consideration of vertical shears and twisting moments at the edge leads to the theorem given by Kelvin and Tait [8] in 1867: The combination of vertical shears and twisting moments at the edge is equivalent to a combination of vertical forces only, in terms of which the reactions are stated; namely, first, a distributed upward reaction,

$$R_y=V_y+\frac{\partial M_{xy}}{\partial x} \qquad (18)$$

secondly, an upward concentrated force at the left end of the edge equal to the value of M_{xy} at that point; and thirdly, a downward concentrated force at the right end of the edge equal to the value of M_{xy} at that point. At an edge parallel to the y-axis, with the slab on the side of the larger values of x one obtains by the same method a distributed upward reaction,

$$R_x=V_x+\frac{\partial M_{xy}}{\partial y} \qquad (19)$$

At a rectangular corner formed by the two edges mentioned, each edge furnishes an upward force equal to the value of M_{xy} at the corner, giving a total concentrated force equal to $2M_{xy}$.

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[8] Thomson (Lord Kelvin) and Tait, Natural Philosophy, 1867. See arts. 645-648 in the later editions.

The problem of the ordinary theory of flexure of the slab is to find a solution of Lagrange's equation 16, satisfying the special conditions existing at the boundary of the area investigated. The boundary conditions are expressed by use of equations 11, 12, 13, 17, 18, and 19.

USE OF INFINITE SERIES EXPLAINED

Consider a simple beam with span s, carrying some concentrated loads and in addition a distributed load, the latter expressed by the function $p=p(x)$, the distance x being measured from the left end. The vertical shear in this beam is a function $V=V(x)$, which changes suddenly at the points of application of the concentrated loads, and which at all other points is governed by the relation,

$$p=-\frac{dV}{dx} \qquad (20)$$

Any function V which is obtainable in this manner may be expressed by a Fourier series, which converges toward V except at the points of application of the concentrated loads and at the ends of the beam, of the form

$$V_1=\sum_{1,2,\ldots}^{n} c_n \cos\frac{n\pi x}{s} \qquad (21)$$

where $c_1,\ c_2,\ \ldots\ c_n,\ \ldots$ are constants. Assuming that a set of constants exists bringing about the convergence,[9] one may determine the constants by the criterion,

$$\int_0^s (V-V_1)\cos\frac{m\pi x}{s}\,dx=0,\ m=1,\ 2,\ \ldots \qquad (22)$$

Using the relations,

$$\int_0^s \cos\frac{m\pi x}{s}\cos\frac{n\pi x}{s}\,dx=\begin{cases}0 \text{ when } n\neq m\\ \frac{s}{2} \text{ when } n=m\end{cases} \qquad (23)$$

one finds, by substituting V_1 from equation 21 in equation 22:

$$c_m=\frac{2}{s}\int_0^s V\cos\frac{m\pi x}{s}\,dx \qquad (24)$$

whereby all the constants $c_1,\ c_2,\ \ldots$ may be determined when the function V is known.

By differentiating equation 21 and reversing signs, one obtains a new Fourier series,

$$p_1=-\frac{dV_1}{dx}=\sum_{1,2,\ldots}^{n}\frac{n\pi c_n}{s}\sin\frac{n\pi x}{s} \qquad (25)$$

which in a special case converges toward p in equation 20 at all points where p does not change suddenly; this special case is that in which all the concentrated loads are zero. If the concentrated loads are not zero, the Fourier series in equation 25 becomes divergent. Yet, integration of the series, with reversal of signs, reproduces V_1 in equation 21, and further successive integrations lead to expressions for the bending moments, slopes, and deflections in terms of convergent Fourier series. So far as these effects are concerned, the aggregation of individual loads $\frac{n\pi c_n}{s}\sin\frac{n\pi x}{s}$ expressed by the divergent series in equation 25, is equivalent to the complete load on the beam. That is, the series in equation 25, in spite of being divergent, represents the complete load on the beam, consisting of the distributed load $p(x)$, and the concentrated forces.[10]

The series in equations 21 and 25 apply outside the interval $0<x<s$ when the function V is periodic with period $2s$, and symmetrical with respect to the points $x=0$ and $x=s$, that is, when $V(x)=V(-x)$, and $V(s+x)=V(s-x)$. The function p_1 has the same period, and is antisymmetrical with respect to the points $x=0$ and $x=s$, that is, $p_1(x)=-p_1(-x)$, $p_1(s+x)=-p_1(s-x)$. The functions apply then to a continuous beam with simple supports at the points $x=0,\ \pm s,\ \pm 2s\ \ldots\ldots$

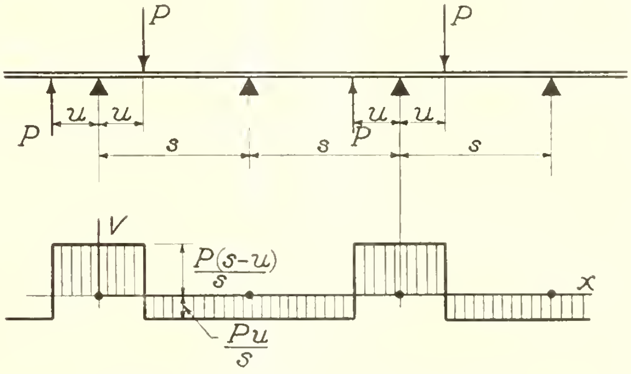

Figure 5.—Vertical Shears in Beam

In the case shown in Figure 5 one finds, using equations 24, 21, and 25, and writing V for V_1, and p for p_1:

$$V=\frac{2P}{\pi}\sum_{1,2,\ldots}^{n}\frac{1}{n}\sin\frac{n\pi u}{s}\cos\frac{n\pi x}{s} \qquad (26)$$

$$p=\frac{2P}{s}\sum_{1,2,\ldots}^{n}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (27)$$

The latter expression will be used in representing a concentrated load on the slab.

SOLUTION FOR SLAB LOADED BY CONCENTRATED FORCE, EXPRESSED BY INFINITE SERIES

The y-axis is placed temporarily at the left edge of the slab. The edges, at $x=0$ and $x=s$, are simply supported. The slab extends infinitely far in the directions of $+y$ and $-y$, and is loaded by a single force P at the point $x=u$, $y=0$. This load will be represented as a load p on the x-axis defined by equation 27.

The function z, representing the deflections in the part of the slab in which y is positive, is defined by the requirement that it must satisfy Lagrange's equation,

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[9] For proof of the existence of the set of constants bringing about the convergence, see, for example, E. T. Whittaker and G. N. Watson, Modern Analysis, second edition (Cambridge), 1915, p. 161.

[10] The use of divergent Fourier series in representing concentrated loads was introduced by A. Mesnager, Comptes Rendus, vol. 164, 1917, p. 600, and has been used extensively by A. Nádai; see his book, Die elastischen Platten. 1925, p. 82.

$\Delta^2 z=0$ (equation 16 with $w=0$), at all points within the area, and in addition the following boundary conditions:

At the edges $x=0$ and $x=s$ and at $y=\infty$:

$$z=\Delta z=0 \qquad (28)$$

At $y=0$:

$$\frac{\partial z}{\partial y}=0 \text{ and } V_y=-\frac{1}{2}p=-\frac{P}{s}\sum_{1,2,\cdots}^{n}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (29)$$

One may determine the function z by a partly deductive process. For the present purpose, it is sufficient however, to state the solution, and then verify it. The following solution [11] satisfies all the requirements:

$$z=\frac{Ps^2}{2\pi^3 N}\sum_{1,2\cdots}^{n}\frac{1}{n^3}\left(1+\frac{n\pi y}{s}\right)e^{-\frac{n\pi y}{s}}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (30)$$

It is seen immediately that $z=0$ for $x=0$ and $x=s$, and for $y=\infty$. One finds, furthermore,

$$\frac{\partial z}{\partial y}=-\frac{P}{2\pi N}\sum_{1,2,\cdots}^{n}\frac{1}{n}\,y e^{-\frac{n\pi y}{s}}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (31)$$

which becomes zero when $y=0$.

$$\frac{\partial^2 z}{\partial y^2}=-\frac{P}{2\pi N}\sum_{1,2,\cdots}^{n}\frac{1}{n}\left(1-\frac{n\pi y}{s}\right)e^{-\frac{n\pi y}{s}}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (32)$$

$$\frac{\partial^2 z}{\partial x^2}=-\frac{P}{2\pi N}\sum_{1,2,\cdots}^{n}\frac{1}{n}\left(1+\frac{n\pi y}{s}\right)e^{-\frac{n\pi y}{s}}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (33)$$

$$\Delta z=\frac{\partial^2 z}{\partial x^2}+\frac{\partial^2 z}{\partial y^2}=-\frac{P}{\pi N}\sum_{1,2,\cdots}^{n}\frac{1}{n}e^{-\frac{n\pi y}{s}}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (34)$$

which becomes zero when $x=0$, $x=s$, or $y=\infty$.

$$V_y=-N\frac{\partial\Delta z}{\partial y}=-\frac{P}{s}\sum_{1,2,\cdots}^{n}e^{-\frac{n\pi y}{s}}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (35)$$

which assumes the required form when $y=0$. Finally, one finds, $\frac{\partial^2\Delta z}{\partial x^2}=-\frac{\partial^2\Delta z}{\partial y^2}$, that is, $\Delta^2 z=0$.

Nádai [12] observed (as may be verified without difficulty) that by introducing the function,

$$\varphi=N\Delta z=-\frac{P}{\pi}\sum_{1,2,\cdots}^{n}\frac{1}{n}e^{-\frac{n\pi y}{s}}\sin\frac{n\pi u}{s}\sin\frac{n\pi x}{s} \qquad (36)$$

one may restate equations 32 and 33 and express $\frac{\partial^2 z}{\partial x\partial y}$ in the following simple form:

$$2N\frac{\partial^2 z}{\partial x^2}=\varphi-y\frac{\partial\varphi}{\partial y} \qquad (37)$$

$$2N\frac{\partial^2 z}{\partial y^2}=\varphi+y\frac{\partial\varphi}{\partial y} \qquad (38)$$

$$2N\frac{\partial^2 z}{\partial x\partial y}=y\frac{\partial\varphi}{\partial x} \qquad (39)$$

Then one finds by equations 11, 12, and 13:

$$M_x=-\frac{1+\mu}{2}\varphi+\frac{1-\mu}{2}y\frac{\partial\varphi}{\partial y} \qquad (40)$$

$$M_y=-\frac{1+\mu}{2}\varphi-\frac{1-\mu}{2}y\frac{\partial\varphi}{\partial y} \qquad (41)$$

$$M_{xy}=-\frac{1-\mu}{2}y\frac{\partial\varphi}{\partial x} \qquad (42)$$

NÁDAI'S SOLUTION IN FINITE FORM PROVED

Nádai,[13] by a deductive process involving functions of a complex variable, derived an expression in finite form for the function φ in equations 36 to 42. Again, it will be sufficient here to state the expression and verify it.

The origin of coordinates is placed now at the center of the span, as in Figure 1. The edges have the equations $x=\pm\frac{s}{2}$, and the point of application of the load P has the coordinates $x=-v=-\frac{s}{2}+u$, $y=0$. The expression found by Nádai, then, is stated as follows:

$$\varphi=N\Delta z=\frac{P}{4\pi}\log_e\frac{B}{A} \qquad (43)$$

where

$$A=\cosh\frac{\pi y}{s}+\cos\frac{\pi(x-v)}{s} \qquad (44)$$

$$B=\cosh\frac{\pi y}{s}-\cos\frac{\pi(x+v)}{s} \qquad (45)$$

The function φ in equation 43 is the same as the function φ in equation 36 (restated in terms of the new coordinates) if it satisfies the following requirements:

First, $\Delta\varphi=0$ at all points except at the point of application of P.

Secondly, $\varphi=0$ at $x=\pm\frac{s}{2}$ and for $y=\infty$.

Third, the total vertical shear at the circumference of a small circle drawn around the load shall be $-P$.

To show that the first requirement is satisfied, the derivatives of φ in equation 43 are expressed. One finds

$$\frac{\partial\varphi}{\partial x}=\frac{P}{4s}\left(\frac{\sin\frac{\pi(x+v)}{s}}{B}+\frac{\sin\frac{\pi(x-v)}{s}}{A}\right) \qquad (46)$$

$$\frac{\partial\varphi}{\partial y}=\frac{P}{4s}\sinh\frac{\pi y}{s}\left(\frac{1}{B}-\frac{1}{A}\right) \qquad (47)$$

Then, by use of the relation, $\cosh^2\frac{\pi y}{s}-\sinh^2\frac{\pi y}{s}=1$, one finds

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[11] A. Nádai, Die elastischen Platten, 1925, p. 85.

[12] A. Nádai, Die elastischen Platten, 1925, p. 86.

[13] A. Nádai, Die elastischen Platten, 1925, p. 89.

$$\left.\begin{array}{r}\dfrac{\partial^2\varphi}{\partial x^2}\\ -\dfrac{\partial^2\varphi}{\partial y^2}\end{array}\right\}=\frac{\pi P}{4s^2}\left[\frac{\cos\dfrac{\pi(x+v)}{s}\cosh\dfrac{\pi y}{s}-1}{B^2}+\frac{\cos\dfrac{\pi(x-v)}{s}\cosh\dfrac{\pi y}{s}+1}{A^2}\right],$$

that is, $\Delta\varphi=\dfrac{\partial^2\varphi}{\partial x^2}+\dfrac{\partial^2\varphi}{\partial y^2}=0.$

The second requirement is satisfied because $A=B$ when $x=\pm\frac{s}{2}$, and because $\frac{B}{A}$ converges toward 1 when y increases indefinitely.

That the third requirement is satisfied, may be shown as follows: When α and β are small values, one may write, $\cos\alpha=1-\frac{\alpha^2}{2}$, $\cosh\beta=1+\frac{\beta^2}{2}$. Consequently, in the immediate neighborhood of the point $x=-v$, $y=0$, where $x+v$ and y are small, equation 45 may be replaced by the simpler expression,

$$B=\frac{\pi^2}{2s^2}(y^2+(x+v)^2)=\frac{\pi^2r^2}{2s^2}\text{ ---------- }(48)$$

where r is the distance between the points $-v$, 0 and x, y. Since $\log_e B$ is numerically large and varies rapidly in this neighborhood while $\log_e A$ varies relatively slowly, one may use for A the value at the point $x=-v$, $y=0$, that is,

$$A=1+\cos\frac{2\pi v}{s}=2\cos^2\frac{\pi v}{s}\text{ ------- }(49)$$

Then equation 43 assumes the following form, applicable when the distance r from the point of application of the load is small:

$$\varphi=\frac{P}{2\pi}\log_e\frac{\pi r}{2s\cos\dfrac{\pi v}{s}}\text{ ------------ }(50)$$

Equations 17, for the vertical shears, may be written: $V_x=-\frac{\partial\varphi}{\partial x}$ $V_y=-\frac{\partial\varphi}{\partial y}$. Correspondingly, the vertical shear in a section perpendicular to the radius vector r may be written: $V_r=-\frac{\partial\varphi}{\partial r}$. Then equation 50 gives $V_r=-\frac{P}{2\pi r}$, that is, the total shear at the circumference of the small circle with radius r is $-P$. Thus all the requirements are satisfied.

Part III.—DERIVATION OF FORMULAS WHICH HAVE DIRECT APPLICATION TO THE PROBLEM OF BRIDGE FLOORS

DETERMINATION OF MOMENTS AT ONE POINT DUE TO A CONCENTRATED LOAD AT ANOTHER POINT

Using equations 40 to 47, one may express the bending moments M_x and M_y and the twisting moment M_{xy} produced at the point x, y by the load P at the point $-v$, 0. One finds

$$\left.\begin{array}{r}M_x\\ M_y\end{array}\right\}=\frac{(1+\mu)P}{8\pi}\log_e\frac{A}{B}\pm\frac{(1-\mu)Py}{8s}\sinh\frac{\pi y}{s}\left(\frac{1}{B}-\frac{1}{A}\right)\text{ - }(51)$$

$$M_{xy}=-\frac{(1-\mu)Py}{8s}\left(\frac{\sin\dfrac{\pi(x-v)}{s}}{A}+\frac{\sin\dfrac{\pi(x+v)}{s}}{B}\right)\text{ - - - }(52)$$

where

$$A=\cosh\frac{\pi y}{s}+\cos\frac{\pi(x-v)}{s},\quad B=\cosh\frac{\pi y}{s}-\cos\frac{\pi(x+v)}{s}.$$

One may use these formulas to obtain expressions for the moments produced at the point $-v$,0 (the point of application of P_1 in Figure 1) by a load P at the point x, y. It is necessary for this purpose to let the points $-v$, 0 and x, y exchange significances. That is, one replaces x, y, and v by $-v$, $-y$, and $-x$, respectively. By this exchange the expressions for A and B remain the same. Denoting the new moments by M'_x, M'_y, and M'_{xy}, one finds

$$M'_x=M_x,\ M'_y=M_y\text{ ------------ }(53)$$

$$M'_{xy}=\frac{(1-\mu)Py}{8s}\left(\frac{\sin\dfrac{\pi(x-v)}{s}}{A}-\frac{\sin\dfrac{\pi(x+v)}{s}}{B}\right)\text{ - }(54)$$

That is, a law of reciprocity applies to the bending moments: The bending moments in the directions of x and y produced at point 1 by a load P at point 2 are the same as those produced at point 2 by a load P at point 1. It becomes unnecessary, therefore, to distinguish between M_x and M'_x, or between M_y and M'_y. The twisting moments, on the other hand, do not follow this law of reciprocity; M'_{xy} differs from M_{xy}.

With $\mu=0.15$, equations 51, 52, and 54 may be written as follows: [14]

$$\left.\begin{array}{r}M_x\\ M_y\end{array}\right\}=0.10536\ P\log_{10}\frac{A}{B}\pm$$

$$0.10625\frac{Py}{s}\sinh\frac{\pi y}{s}\left(\frac{1}{B}-\frac{1}{A}\right)\text{ ---------- }(55)$$

$$\left.\begin{array}{r}M_{xy}\\ M'_{xy}\end{array}\right\}=-0.10625\frac{Py}{s}\left(\frac{\sin\dfrac{\pi(x+v)}{s}}{B}\pm\frac{\sin\dfrac{\pi(x-v)}{s}}{A}\right)\text{ - }(56)$$

EFFECTS OF LOAD DISTRIBUTED UNIFORMLY OVER THE AREA OF A SMALL CIRCLE

Consider now a load P which is distributed uniformly over the area of a small circle with center at the point $-v$, 0 and with the diameter c, as P_1 in Figure 1. In order to obtain the correct maximum tensile stress at the bottom of the slab by use of the ordinary theory of flexure, the moments will be determined (as proposed in the introduction) as if the load were distributed uniformly over the area of a circle with diameter c_1 instead of c.[15]

By using polar coordinates r, θ, with the pole at the center of the circle, and with the angle θ measured from the x-axis, the load on an element of the area of the circle will be expressed as $\frac{4P}{\pi c_1^2} r dr d\theta$. On account of the reciprocal relation of bending moments (equations 53), the bending moments produced at the center of the circle may be computed by means of equation 51. Since the distances are small, the values of A and B may be taken from equations 49 and 48, respectively. The term $\frac{1}{A}$ at the end of equation 51 may be ignored as insignificant in comparison with $\frac{1}{B}$. Moreover, $\sinh \frac{\pi y}{s}$ may be replaced by $\frac{\pi y}{s}$. Then equation 51 leads to the following values of the resultant moments at the center of the circle:

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[14] Numerical computations based on these equations are made conveniently by use of the tables published by K. Hayashi, Sieben- und mehrstellige Tafeln der Kreis- und Hyperbelfunktionen und deren Produkte sowie der Gammafunktion, (Berlin), 1926.

[15] See footnote 3 on p. 2 and the explanation following this reference.

$$\left.\begin{matrix} M_x \\ M_y \end{matrix}\right\} = \int_{-\pi}^{\pi} d\theta \int_0^{\frac{c_1}{2}} \frac{4P}{\pi c_1^2} r dr \left(\frac{1+\mu}{4\pi} \log_e \frac{2s \cos \frac{\pi v}{s}}{\pi r} \pm \frac{(1-\mu)\sin^2\theta}{4\pi} \right).$$

Since $\int_0^{\frac{c_1}{2}} r dr \log_e r = \frac{c_1^2}{8} \left(\log_e \frac{c_1}{2} - \frac{1}{2} \right)$, one finds

$$\left.\begin{matrix} M_x \\ M_y \end{matrix}\right\} = \frac{(1+\mu)P}{4\pi} \left(\log_e \left(\frac{4s}{\pi c_1} \cos \frac{\pi v}{s} \right) + \frac{1}{2} \right) \pm \frac{(1-\mu)\,P}{8\pi} \quad \text{(57)}$$

The equivalent diameter c_1 is expressed with satisfactory approximation by the following formula,[16] applicable when $c < 3.45h$:

$$c_1 = 2(\sqrt{0.4c^2 + h^2} - 0.675\,h) \quad \text{(58)}$$

GREATEST BENDING MOMENTS COMPUTED FOR CASE OF WHEEL LOAD AT CENTER

When the load is at the center, that is, $v = 0$, the moments M_x and M_y in equation 57 assume the following values, which are denoted by M_{0x} and M_{0y}, respectively:

$$M_{0x} = \frac{P}{4\pi} \left((1+\mu) \log_e \frac{4s}{\pi c_1} + 1 \right) \quad \text{(59)}$$

$$M_{0y} = M_{0x} - \frac{(1-\mu)P}{4\pi} \quad \text{(60)}$$

or, with $\mu = 0.15$, and c_1 substituted from equation 58:

$$M_{0x} = 0.21072\,P \left[\log_{10} \frac{s}{h} - \log_{10} \left(\sqrt{0.4 \frac{c^2}{h^2} + 1} - 0.675 \right) + 0.1815 \right] \quad \text{(61)}$$

$$M_{0y} = M_{0x} - 0.0676\;P \quad \text{(62)}$$

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[16] Equation 8 in the paper by the writer, Stresses in Concrete Pavements Computed by Theoretical Analysis, Public Roads, vol. 7, No. 2, April, 1926.

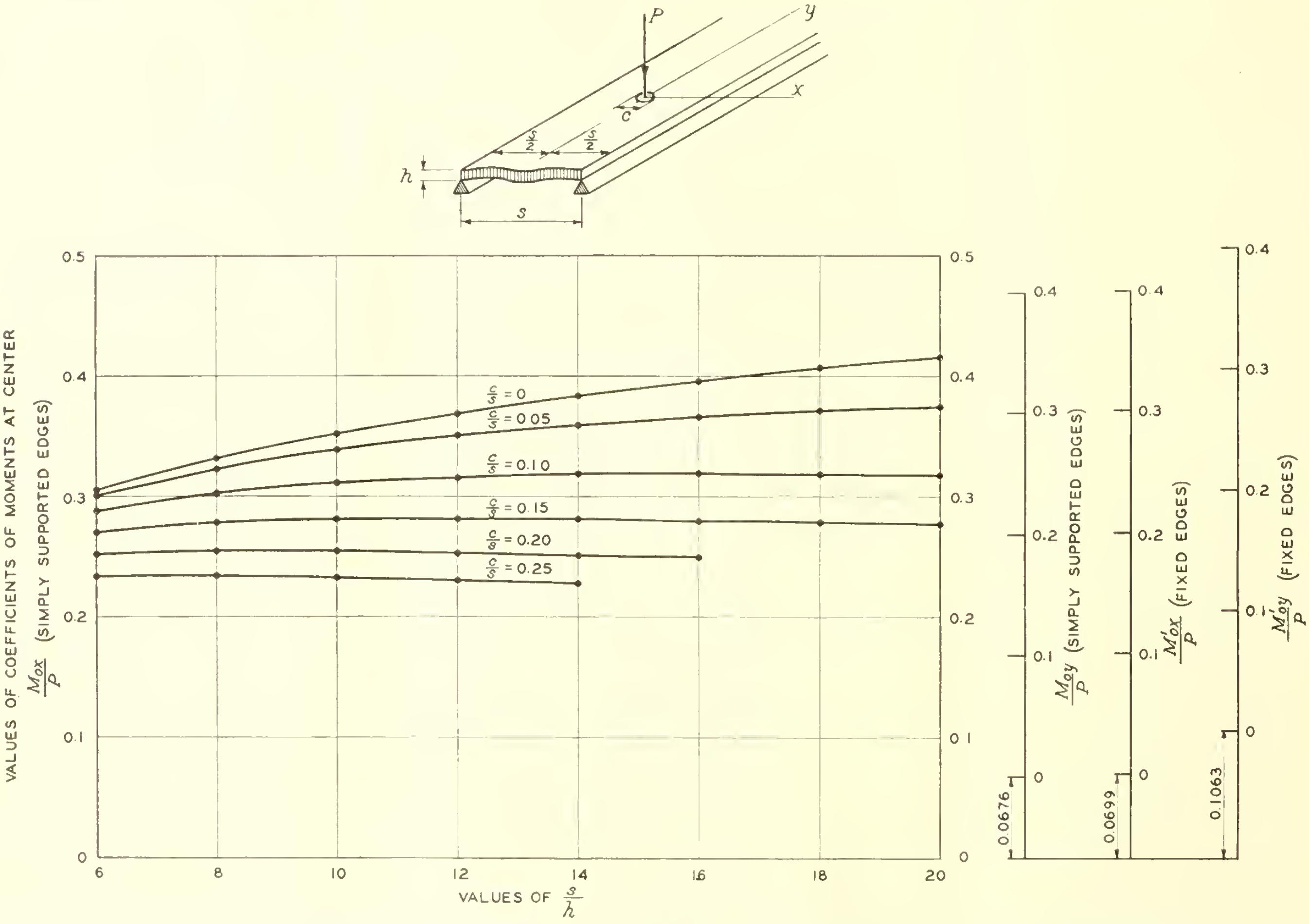

FIGURE 6.—COEFFICIENTS OF BENDING MOMENTS, M_{0x} AND M_{0y}, IN DIRECTIONS OF x AND y, RESPECTIVELY, PRODUCED AT CENTER OF SLAB BY A CENTRAL LOAD P DISTRIBUTED UNIFORMLY OVER THE AREA OF A SMALL CIRCLE WITH DIAMETER c. RESULTS REPRESENTING EQUATIONS 61, 62, 104, AND 105. NUMERICAL VALUES STATED IN TABLE 1. POISSON'S RATIO, $\mu = 0.15$

TABLE 1.—*Values of the coefficient $\frac{M_{0x}}{P}$ of the maximum bending moment per unit of width, produced at the center of the slab in the direction of the span by a central load P distributed uniformly over the area of a small circle with diameter c. The edges are assumed to be simply supported. The values were computed from equation 61 for different relative values of the span, s, the thickness, h, and the diameter c. Figure 6 shows the results graphically. Poisson's ratio, $\mu=0.15$*

	$c=0$	$c=0.05s$	$c=0.10s$	$c=0.15s$	$c=0.20s$	$c=0.25s$
$s=6h$	0.3051	0.3003	0.2874	0.2701	0.2520	0.2345
$s=8h$	.3315	.3230	.3026	.2784	.2552	.2345
$s=10h$	.3519	.3390	.3110	.2811	.2550	.2326
$s=12h$	.3685	.3508	.3154	.2815	.2535	.2303
$s=14h$	.3827	.3595	.3178	.2809	.2516	.2284
$s=16h$	.3949	.3660	.3186	.2798	.2499	----------
$s=18h$	.4056	.3709	.3186	.2786	----------	----------
$s=20h$	.4153	.3744	.3184	.2771	----------	----------

Table 1 and Figure 6 show values of the coefficient $\frac{M_{0x}}{P}$ computed from equation 61. The coefficients stated are pure numbers. If, for example, one reads in Figure 6, $\frac{M_{0x}}{P}=0.3$, the significance is: $M_{0x}=0.3\ P$, or, with $P=10{,}000$ pounds, $M_{0x}=0.3\times10{,}000$ pounds $=3{,}000$ pounds $=3{,}000\ \frac{\text{in. lbs.}}{\text{in.}}=3{,}000\ \frac{\text{ft. lbs.}}{\text{ft.}}$ (the unit of bending moment per unit of width being inch-pounds per inch or foot-pounds per foot or simply pounds). If units of the metric system were used, the coefficients in Figure 6 would remain unchanged. These comments apply also to the coefficients stated in the diagrams and tables which are given later.

Since the difference between $\frac{M_{0x}}{P}$ and $\frac{M_{0y}}{P}$ is constant, the curves in Figure 6 also represent values of $\frac{M_{0y}}{P}$, on a separate scale. The third scale from the right serves this purpose.

The moment M_{0x} could be produced as the maximum moment per unit of width in a simple beam with span s and width b_e, the load P being applied at the center of the span, and distributed over the width of the beam. Ignoring the effects of Poisson's ratio, one may assume the bending moment to be distributed uniformly over the width. The width b_e bringing about this equivalence of a slab and a beam is called the effective width.[17] It is defined by the equation,

$$M_{0x}=\frac{1}{4}\frac{P}{b_e}s \qquad (63)$$

or,

$$b_e=\frac{Ps}{4M_{0x}} \qquad (64)$$

Values of b_e, computed from this equation, with M_{0x} defined by equation 61, are shown in Table 2 and Figure 7. Knowing b_e one may compute the bending moments by equation 63.

The diagram at the right of Figure 7 shows a set of straight horizontal lines which may be allowed to take the place of the curves in a crude, approximate computation. To be on the side of safety the straight lines should be drawn so as to represent the low values rather than the average values defined by the curves. The straight lines are drawn according to the formula,

$$b_e=0.58s+2c \qquad (65)$$

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[17] E. F. Kelley, Effective Width of Concrete Bridge Slabs Supporting Concentrated Loads, Public Roads, vol. 7, No. 1, March, 1926, p. 7.

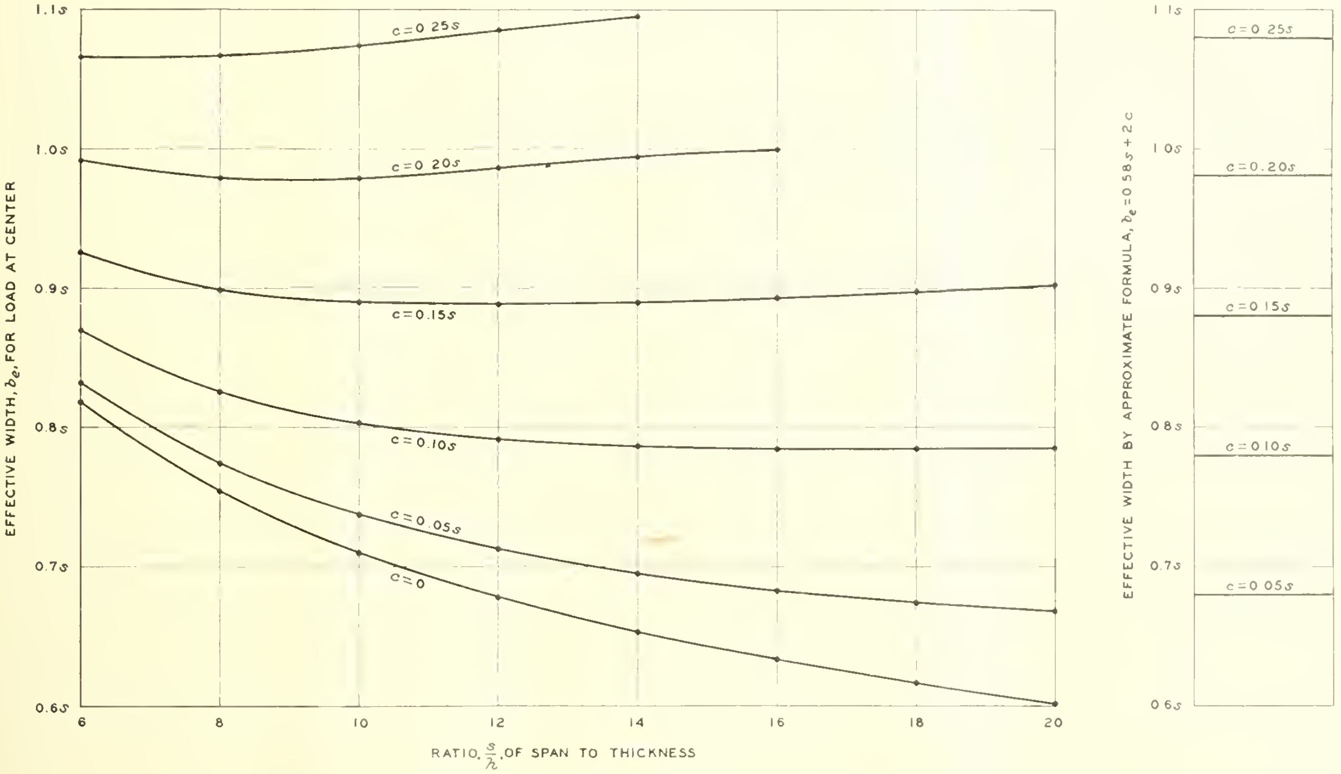


FIGURE 7.—EFFECTIVE WIDTH b_e FOR CENTRAL LOAD, DISTRIBUTED UNIFORMLY OVER THE AREA OF A SMALL CIRCLE WITH DIAMETER c, WHEN THE EDGES ARE SIMPLY SUPPORTED (FROM EQUATIONS 64 AND 65, AND TABLE 2). POISSON'S RATIO, $\mu=0.15$

A corresponding, roughly approximate expression for the bending moment is obtained by substituting this value of b_e in equation 63:

$$M_{0x}=\frac{Ps}{2.32s+8c} \quad \text{--------------(66)}$$

TABLE 2.—*Values of the ratio, $\frac{b_e}{s}$, of the effective width to the span, in the cases represented in Table 1. The values were computed from equations 64 and 61, and are represented graphically in Figure 7. Poisson's ratio, $\mu=0.15$*

	$c=0$	$c=0.05s$	$c=0.10s$	$c=0.15s$	$c=0.20s$	$c=0.25s$
$s=6h$	0.819	0.832	0.870	0.925	0.992	1.066
$s=8h$	.754	.774	.826	.898	.980	1.066
$s=10h$	.710	.737	.804	.890	.980	1.075
$s=12h$	.678	.713	.792	.888	.986	1.085
$s=14h$	.653	.695	.787	.890	.994	1.095
$s=16h$	.633	.683	.785	.893	1.000	---------
$s=18h$	.616	.674	.785	.897	---------	---------
$s=20h$	.602	.668	.785	.902	---------	---------

MOMENTS COMPUTED FOR CASE OF TWO WHEEL LOADS ON LINE IN DIRECTION OF SPAN

Figure 8 (a) shows the case of two wheel loads, P_1 at the point 0, 0, and P_2 at the point x, 0. The effects produced by P_1 at the point of application of P_1 are expressed by equations 59 and 61, and are represented in Table 1 and Figure 6. The moments contributed at the point of application of P_1 in Figure 8 (a) by P_2 may be obtained from equation 51. One finds

$$\frac{A}{B}=\frac{1+\cos\frac{\pi x}{s}}{1-\cos\frac{\pi x}{s}}=\cot^2\frac{\pi x}{2s},$$

and consequently,

$$M_x=M_y=\frac{(1+\mu)P_2}{4\pi}\log_e\cot\frac{\pi x}{2s} \quad \text{-------(67)}$$

or, with $\mu=0.15$,

$$M_x=M_y=0.21072\ P_2\log_{10}\cot\frac{\pi x}{2s} \quad \text{-------(68)}$$

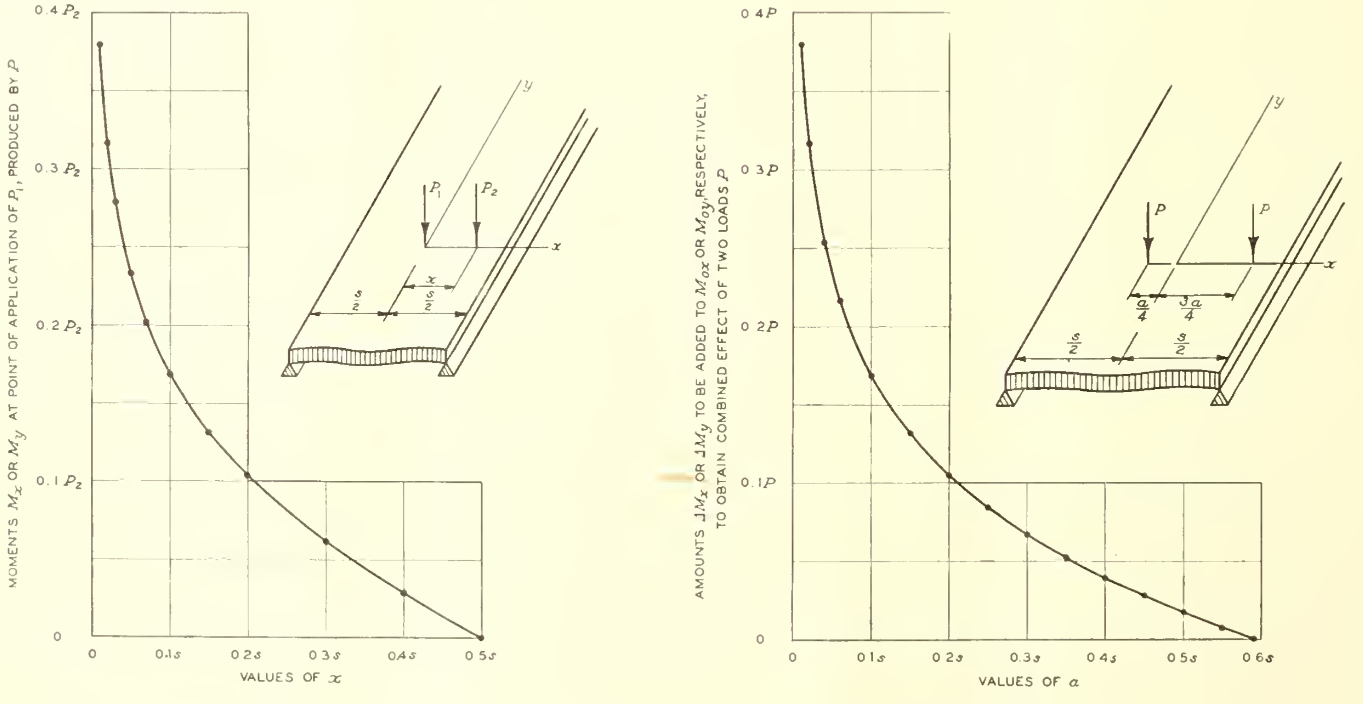
Table 3 and the curve in Figure 8 (a) represent values computed from equation 68.

TABLE 3.—*Coefficients $\frac{M_x}{P_2}$ and $\frac{M_y}{P_2}$ of the moments produced at the point 0, 0 by the load P_2 at the point x, 0, computed from equation 68, and represented graphically in Figure 8 (a). Poisson's ratio, $\mu=0.15$*

$\frac{x}{s}$	$\frac{M_x}{P_2}=\frac{M_y}{P_2}$	$\frac{x}{s}$	$\frac{M_x}{P_2}=\frac{M_y}{P_2}$
0.01	0.3801	0.15	0.1306
.02	.3166	.20	.1029
.03	.2795	.30	.0617
.05	.2326	.40	.0292
.07	.2017	.50	0
.10	.1686		

When there is a fixed distance a between the two loads, the moments under P_1 may be increased by moving the loads toward the left, into the positions of P_1 and P_2 in Figure 1. With P_1 and P_2 at the points $-v$, 0 and $a-v$, 0, respectively, and $P_1=P_2=P$, the moments under P_1 may be expressed as follows, by use of equations 57, 59, 51, and 60:

$$M_x=M_{0x}+\frac{(1+\mu)P}{4\pi}\log_e\cos\frac{\pi v}{s}+\frac{(1+\mu)P}{8\pi}\log_e\frac{1+\cos\frac{\pi(a-2v)}{s}}{1-\cos\frac{\pi a}{s}} \quad \text{------(69)}$$



(a) (b)

FIGURE 8.—BENDING MOMENTS PRODUCED AT POINT OF APPLICATION OF LEFT OF TWO LOADS (FROM EQUATIONS 68 AND 72, AND TABLES 3 AND 4). POISSON'S RATIO, $\mu=0.15$

$$M_y = M_x - \frac{(1-\mu)P}{4\pi} \quad \text{-------------(70)}$$

Since $1+\cos\frac{\pi(a-2v)}{s} = 2\cos^2\frac{\pi(a-2v)}{2s}$, these moments reach their maximum values when the product $f=\cos\frac{\pi v}{s}\cos\frac{\pi(a-2v)}{2s}$, becomes a maximum. By writing $f=\frac{1}{2}\left(\cos\frac{\pi(a-4v)}{2s}+\cos\frac{\pi a}{2s}\right)$, one finds that the condition, $\frac{df}{dv}=0$, gives $\sin\frac{\pi(a-4v)}{2s}=0$, or $v=\frac{a}{4}$. That is, the two equal loads are placed as they would be on a beam. With $v=\frac{a}{4}$, equation 69 becomes,

$$M_x = M_{0x} + \frac{(1+\mu)P}{4\pi}\log_e\frac{\cot\frac{\pi a}{4s}}{2} \quad \text{------(71)}$$

or, with $\mu=0.15$,

$$\left.\begin{array}{l}\frac{\Delta M_x}{P}=\frac{M_x-M_{0x}}{P}\\ \frac{\Delta M_y}{P}=\frac{M_y-M_{0y}}{P}\end{array}\right\} = 0.21072\log_{10}\frac{\cot\frac{\pi a}{4s}}{2} \quad \text{--(72)}$$

These values become negative when $a>0.5903s$. In this case the greatest effect is produced by P_1 alone, placed at the center of the span.

Table 4 and Figure 8 (b) show values computed from equation 72.

TABLE 4.—*Values* $\frac{\Delta M_x}{P}$ *or* $\frac{\Delta M_y}{P}$ *to be added to* $\frac{M_{0x}}{P}$ *(given in Table 1) or* $\frac{M_{0y}}{P}$ *(equation 62), respectively, to obtain the values of* $\frac{M_x}{P}$ *or* $\frac{M_y}{P}$ *due to the combined action of two loads P placed at the points* $-\frac{a}{4}$, *0 and* $\frac{3a}{4}$, *0, computed from equation 72, and represented graphically in Figure 8* (b). *Poisson's ratio,* $\mu=0.15$

$\frac{a}{s}$	$\frac{\Delta M_x}{P}=\frac{\Delta M_y}{P}$	$\frac{a}{s}$	$\frac{\Delta M_x}{P}=\frac{\Delta M_y}{P}$
0.01	0.3801	0.30	0.0671
.02	.3167	.35	.0524
.04	.2532	.40	.0394
.06	.2161	.45	.0278
.10	.1692	.50	.0172
.15	.1319	.55	.0071
.20	.1052	.5903	0
.25	.0844		

MOMENTS COMPUTED FOR TWO LOADS ON CENTER LINE

Figure 9 shows two loads, P_1 at the point 0, 0, and P_3 at the point 0, y. The moments produced under P_1 by P_1 are given by equations 59 to 62 and in Table 1 and Figure 6. To these moments must be added the moments M_x and M_y contributed at the point of application of P_1 by P_3. Equation 51, with $x=v=0$, that is, with

$$A=\cosh\frac{\pi y}{s}+1=2\cosh^2\frac{\pi y}{2s},$$

$$B=\cosh\frac{\pi y}{s}-1=2\sinh^2\frac{\pi y}{2s},$$

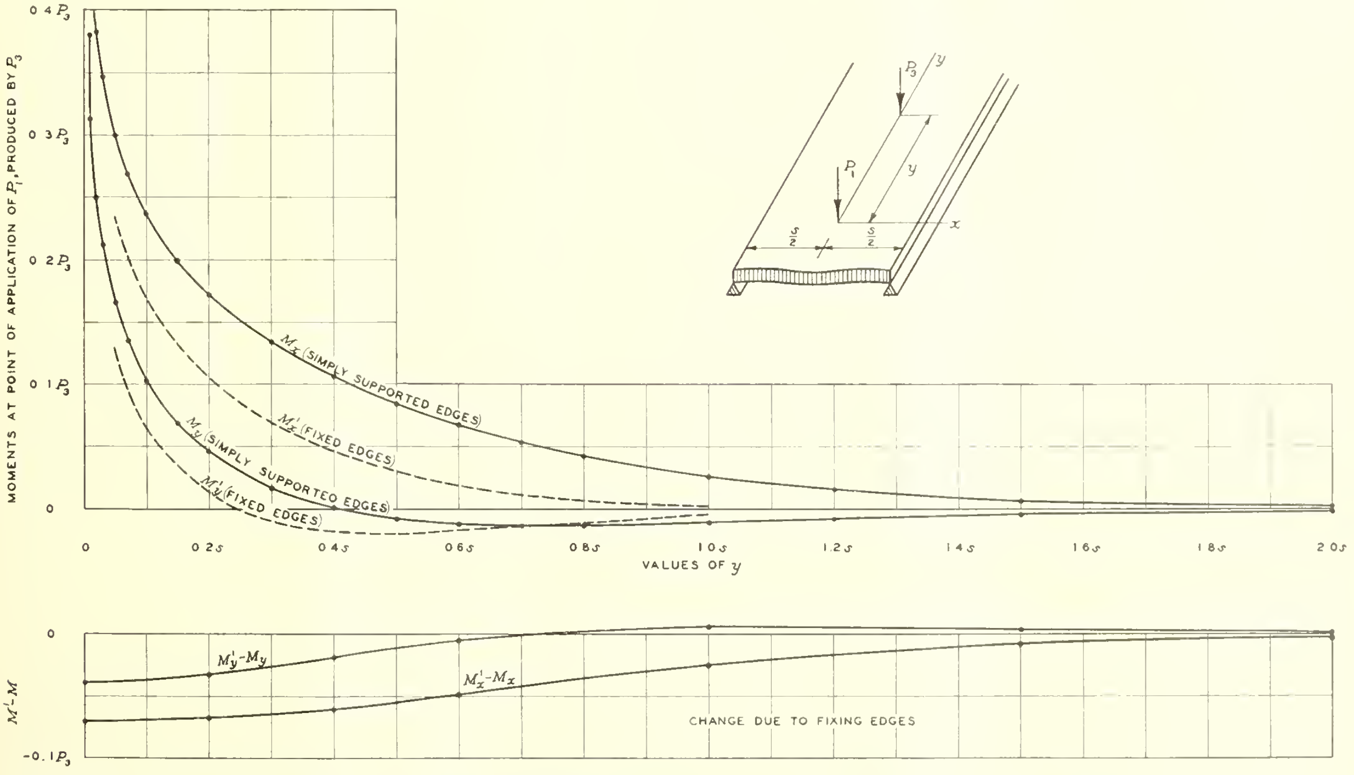


FIGURE 9.—BENDING MOMENTS PRODUCED AT POINT OF APPLICATION OF P_1 BY P_3 (FROM EQUATIONS 74, 102, AND 103, AND TABLES 5 AND 7). POISSON'S RATIO, $\mu=0.15$

and with $P=P_3$, gives

$$\left.\begin{matrix}M_x\\ M_y\end{matrix}\right\}=\frac{(1+\mu)P_3}{4\pi}\log_e\coth\frac{\pi y}{2s}\pm\frac{(1-\mu)P_3y}{4s\sinh\frac{\pi y}{s}}\quad\text{- - - - (73)}$$

or, with $\mu=0.15$,

$$\left.\begin{matrix}M_x\\ M_y\end{matrix}\right\}=0.21072P_3\log_{10}\coth\frac{\pi y}{2s}\pm\frac{0.2125P_3y}{s\sinh\frac{\pi y}{s}}\quad\text{- - - (74)}$$

Coefficients $\frac{M_x}{P_3}$ and $\frac{M_y}{P_3}$, computed from equation 74, are stated in the first section of Table 5. Equation 74 is represented graphically by the curves drawn with full lines in the upper part of Figure 9.

MOMENTS COMPUTED AT CENTER FOR LOAD AT ANY POINT, AND ALSO AT ANY POINT FOR LOAD AT CENTER

Table 5 and Figures 8 to 14 show moments produced at points x, y by a load, $P=1$, at the center, point 0, 0, and moments produced at point 0, 0 by a load, $P=1$, at points x, y, for Poisson's ratio, $\mu=0.15$. All of these moments are defined by equations 44, 45, 55, 56, with $v=0$. Equations 68 and 74 apply to the special cases of $y=0$ and $x=0$, respectively. With $v=0$, the equations for the twisting moments (equations 52 and 54, or equation 56 when $\mu=0.15$) may be written in the simpler forms,

$$M_{xy}=-\frac{(1-\mu)Py}{2s}\,\frac{\sin\frac{\pi x}{s}\cosh\frac{\pi y}{s}}{\cosh\frac{2\pi y}{s}-\cos\frac{2\pi x}{s}}\quad\text{- - - - (75)}$$

$$M'_{xy}=-\frac{(1-\mu)\,Py}{4s}\,\frac{\sin\frac{2\pi x}{s}}{\cosh\frac{2\pi y}{s}-\cos\frac{2\pi x}{s}}\quad\text{- - - (76)}$$

In the special case, $x=\frac{s}{2}$, that is, at the edge, these equations give

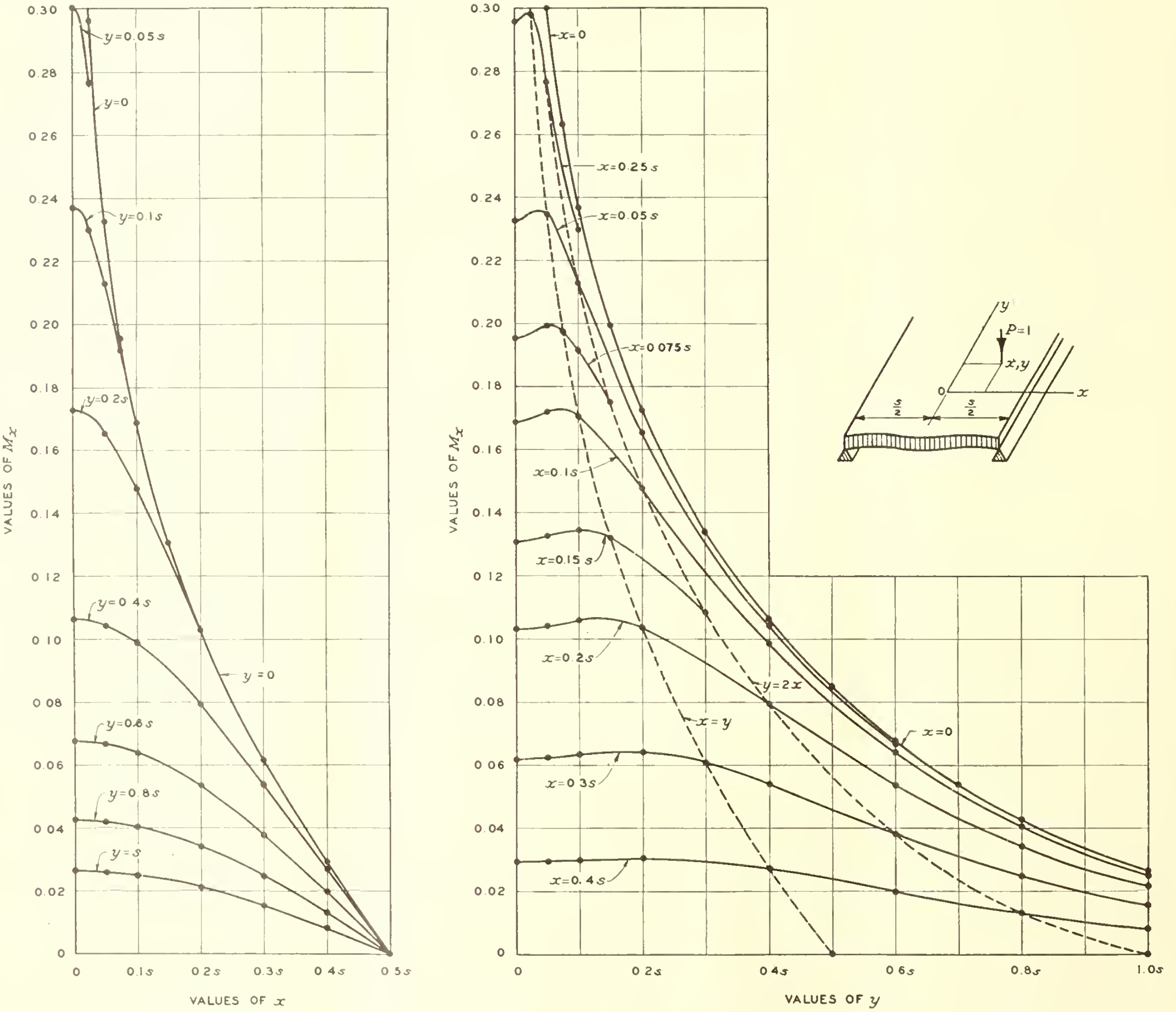


FIGURE 10.—BENDING MOMENTS M_x, PRODUCED AT POINT x, y BY LOAD $P=1$ AT 0, OR AT 0 BY LOAD $P=1$ AT POINT x, y (FROM EQUATION 55 AND TABLE 5). POISSON'S RATIO, $\mu=0.15$

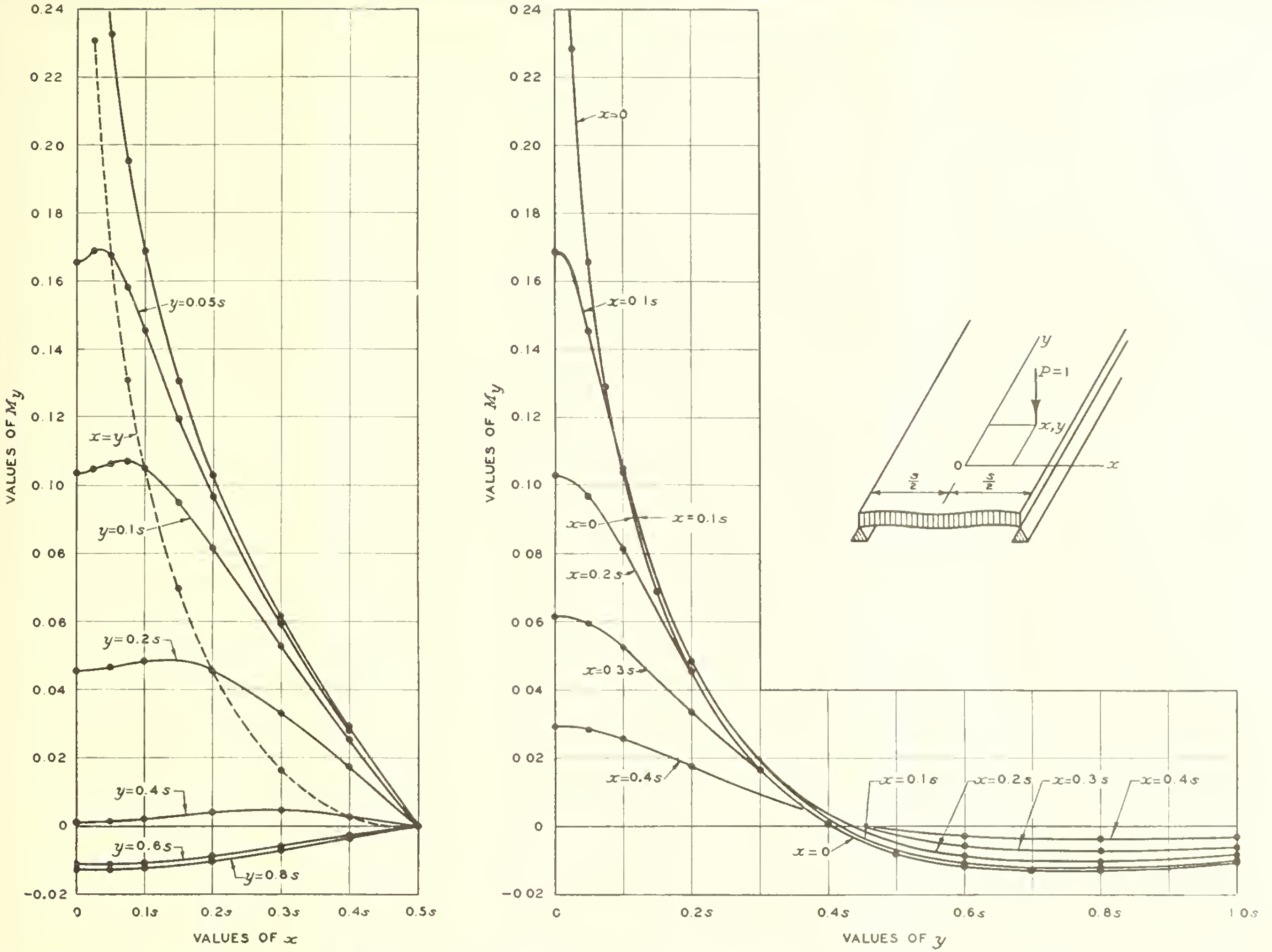


FIGURE 11.—BENDING MOMENTS M_y, PRODUCED AT POINT x, y BY LOAD $P=1$ AT 0, OR AT 0 BY LOAD $P=1$ AT POINT x, y (FROM EQUATION 55 AND TABLE 5). POISSON'S RATIO, $\mu=0.15$

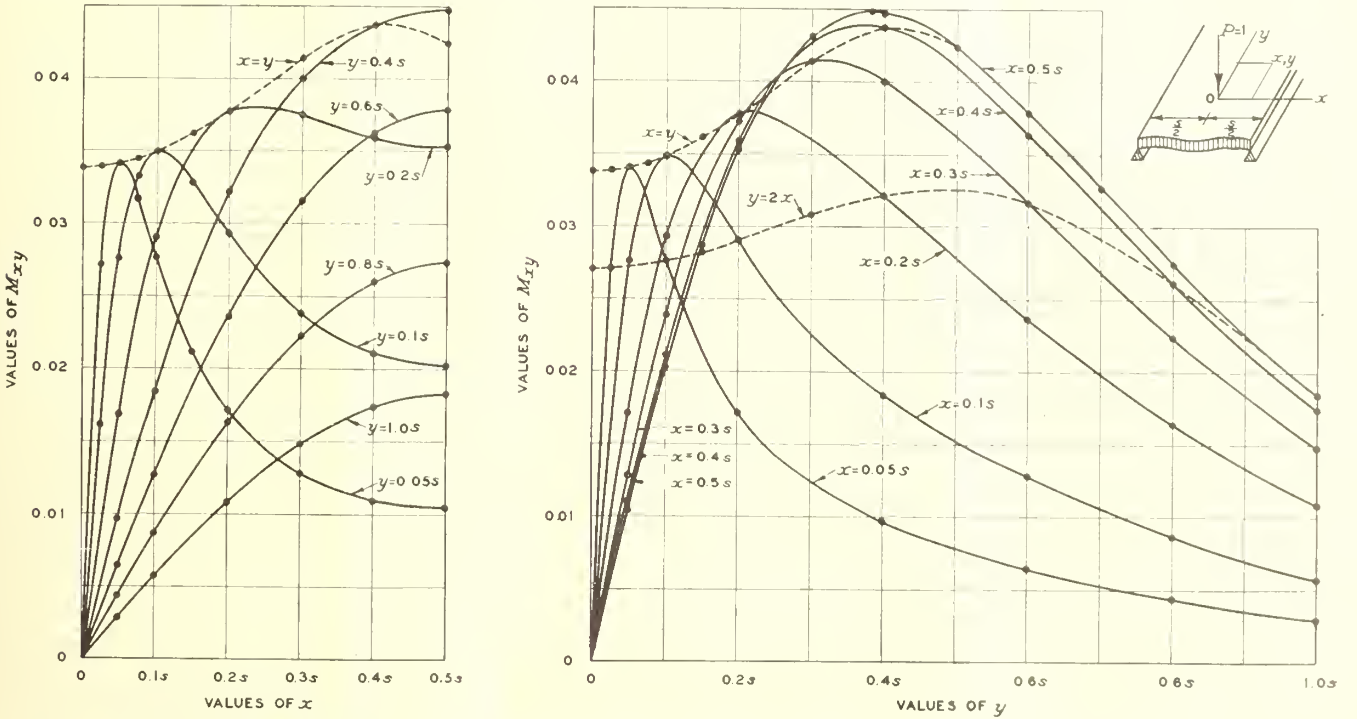


FIGURE 12.—TWISTING MOMENTS M_{xy}, PRODUCED AT POINT x, y BY LOAD $P=1$ AT 0 (FROM EQUATIONS 56, 75, AND 77, AND TABLE 5). POISSON'S RATIO, $\mu=0.15$

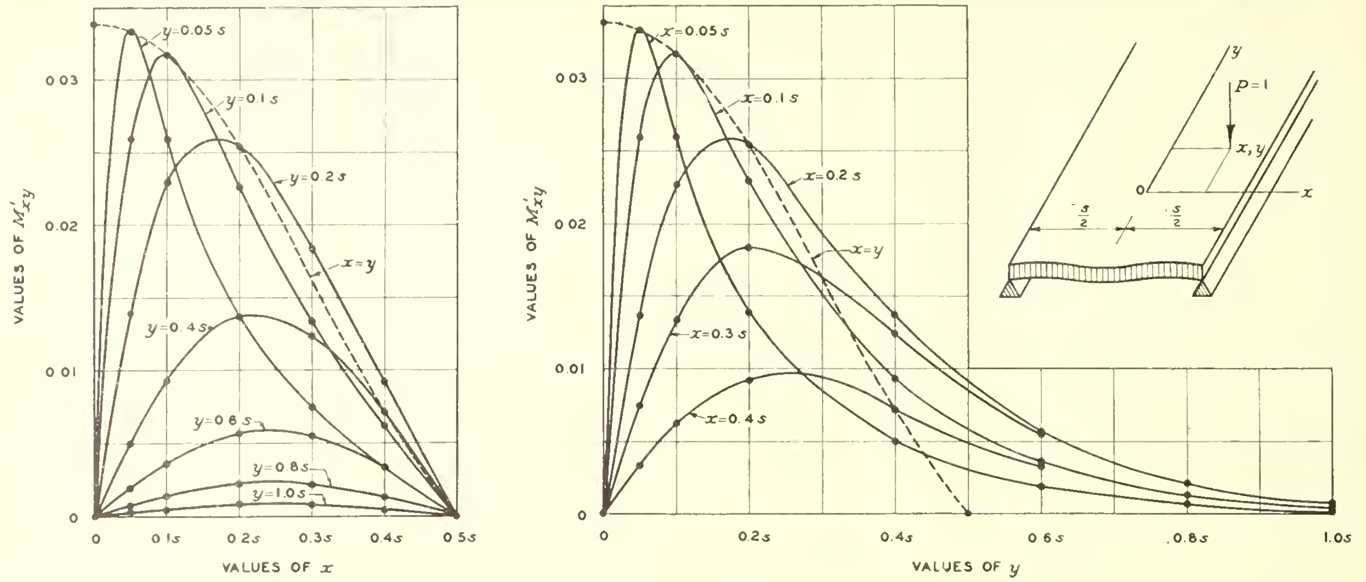


Figure 13.—Twisting Moments M'_{xy}, Produced at 0 by Load $P=1$ at Point x, y (from Equations 56 and 76, and Table 5). Poisson's Ratio, $\mu=0.15$

Table 5.—*Bending moments M_x and M_y produced at point x, y by load $P=1$, at point 0, 0, or at point 0, 0 by load $P=1$ at point x, y, computed from equations 44, 45, 55 (with $v=0$), 68 and 74, and represented graphically in Figures 8* (a), *9, 10, 11, and 14. Twisting moments M_{xy} produced at point x, y by load $P=1$ at point 0, 0, and twisting moments M'_{xy} produced at point 0, 0 by load $P=1$ at point x, y, computed from equations 44, 45, 56 (with $v=0$), 75, 76, and 77, and represented graphically in Figures 12, 13, and 14. Poisson's ratio, $\mu=0.15$*

$\frac{x}{s}$	$\frac{y}{s}$	M_x	M_y	$-M_{xy}$	$-M'_{xy}$
0	0.01	0.448	0.313	0	0
	.02	.384	.249	0	0
	.025	.3639	.2287	0	0
	.03	.347	.212	0	0
	.05	.3004	.1657	0	0
	.07	.269	.135	0	0
	.075	.2632	.1291	0	0
	.1	.2367	.1036	0	0
	.15	.1992	.0688	0	0
	.2	.1723	.0455	0	0
	.3	.1339	.0167	0	0
	.4	.1062	.0009	0	0
	.5	.0848	−.0076	0	0
	.6	.0676	−.0116	0	0
	.7	.0538	−.0130	0	0
	.8	.0426	−.0129	0	0
	1.0	.0263	−.0105	0	0
	1.2	.0160	−.0076	0	0
	1.5	.0074	−.0041	0	0
	2.0	.0019	−.0013	0	0
	2.5	.0005	−.0003	0	0
	3.0	.0001	−.0001	0	0
.01	0	.3801	.3801	0	0
.02	0	.3166	.3166	0	0
.025	0	.2962	.2962	0	0
	.025	.2983	.2308	.03389	----------
	.05	.2766	.1689	.02719	----------
	.1	.2299	.1048	.01619	----------
.03	0	.2795	.2795	0	0
.05	0	.2326	.2326	0	0
	.05	.2346	.1676	.03410	.03327
	.1	.2128	.1067	.02760	.02596
	.2	.1653	.0466	.01689	.01385
	.4	.1043	.0012	.00959	.00499
	.6	.0667	−.0114	.00648	.00190
	.8	.0420	−.0127	.00439	.00069
	1.0	.0260	−.0104	.00289	.00025
.07	0	.2017	.2017	0	0
.075	0	.1953	.1953	0	0
	.05	.1992	.1581	.03164	----------
	.075	.1972	.1308	.03444	----------
	.1	.1915	.1071	.03328	----------
	.15	.1750	.0717	.02822	----------
.1	0	.1686	.1686	0	0
	.05	.1719	.1454	.02761	.02594
	.1	.1704	.1050	.03491	.03163
	.2	.1478	.0483	.02902	.02292
0.1	0.4	0.0987	0.0021	0.01846	0.00925
	.6	.0639	−.0108	.01271	.00359
	.8	.0404	−.0122	.00866	.00132
	1.0	.0250	−.0100	.00571	.00047
.15	0	.1306	.1306	0	0
	.05	.1324	.1195	.02114	----------
	.1	.1342	.0949	.03287	----------
	.15	.1319	.0695	.03619	----------
	.3	.1083	.0194	.03076	----------
.2	0	.1029	.1029	0	0
	.05	.1040	.0967	.01707	.01364
	.1	.1058	.0813	.02930	.02258
	.2	.1035	.0455	.03783	.02542
	.4	.0793	.0040	.03214	.01369
	.6	.0535	−.0086	.02361	.00567
	.8	.0342	−.0102	.01636	.00214
	1.0	.0213	−.0084	.01083	.00075
.3	0	.0617	.0617	0	0
	.05	.0622	.0593	.01281	.00744
	.1	.0632	.0526	.02386	.01336
	.2	.0640	.0337	.03749	.01831
	.3	.0607	.0164	.04145	----------
	.4	.0540	.0046	.04005	.01239
	.6	.0380	−.0058	.03157	.00551
	.8	.0247	−.0073	.02233	.00211
	1.0	.0154	−.0061	.01487	.00075
.1	0	.0292	.0292	0	0
	.05	.0294	.0283	.01101	.00336
	.1	.0298	.0257	.02108	.00621
	.2	.0305	.0175	.03594	.00922
	.4	.0271	.0029	.04373	.00711
	.6	.0197	−.0029	.03629	.00333
	.8	.0129	−.0038	.02608	.00130
	1.0	.0081	−.0032	.01745	.00047
.5	0	0	0	0	0
	.05	0	0	.01049	0
	.1	0	0	.02024	0
	.15	0	0	.02864	0
	.2	0	0	.03530	0
	.3	0	0	.04313	0
	.382	0	0	.04483	0
	.4	0	0	.04476	0
	.5	0	0	.04234	0
	.6	0	0	.03785	0
	.7	0	0	.03259	0
	.8	0	0	.02736	0
	.0	0	0	.01833	0
	11.2	0	0	.01175	0
	1.5	0	0	.00573	0
	2.0	0	0	.00159	0

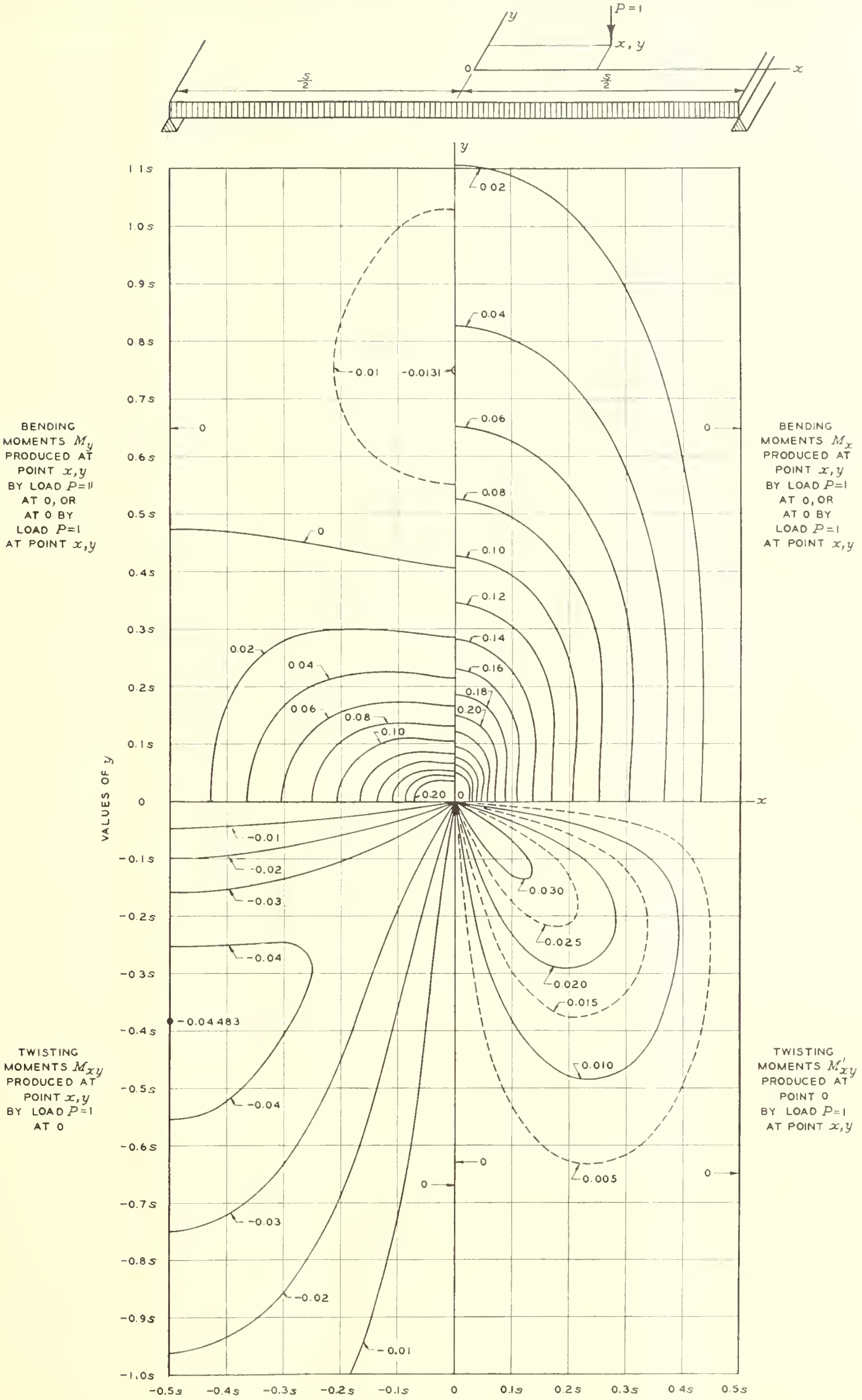


Figure 14.– Contour Lines of Surfaces Representing Moments (Compare Figures 10 to 13). Poisson's Ratio, $\mu = 0.15$

$$M_{xy}=-\frac{(1-\mu)\,Py}{4s\cosh\frac{\pi y}{s}}\text{------------}(77)$$

and $M'_{xy}=0$.

For small values of x and y, that is, in the immediate neighborhood of the point 0, 0, one may write

$$\sin\frac{2\pi x}{s}=2\sin\frac{\pi x}{s}=\frac{2\pi x}{s},\ \cosh\frac{\pi y}{s}=1,\text{ and}$$

$$\cosh\frac{2\pi y}{s}-\cos\frac{2\pi x}{s}=\frac{2\pi^2}{s^2}(y^2+x^2).$$

Then equations 75 and 76 may be written

$$M_{xy}=M'_{xy}=-\frac{(1-\mu)\,P}{4\pi}\frac{xy}{x^2+y^2}=-\frac{(1-\mu)\,P}{8\pi}\sin 2\theta\text{--}(78)$$

where θ is the angle between the x-axis and the radius vector to the point x, y; or, with $\mu=0.15$,

$$M_{xy}=M'_{xy}=-0.06764\,P\frac{xy}{x^2+y^2}$$
$$=-0.03382\,P\sin 2\theta\text{------------}(79)$$

With $x=y$, equation 79 gives $M_{xy}=M'_{xy}=-0.03382P$. With $x=2y$ or $y=2x$, the same equation gives $M_{xy}=M'_{xy}=-0.02706P$.

Attention is called especially to Figure 14, showing contour lines of the surfaces representing the moments.

COMPUTATION OF MOMENTS PRODUCED AT POINT OF APPLICATION OF P_1 IN FIGURE 1 BY THE TWO LOADS P_3 AND P_4

The two loads P_3 and P_4 in Figure 1 will be assumed to be equal, each equal to P. In order to determine the value of v at which the bending moments produced at the point $-v$, 0 by the two loads become as large as possible, the following conditions are introduced temporarily: $P=1$, $s=\pi$, y-axis at the left edge. Then equations 32 and 33 lead to an expression of the following form for the bending moment M_x produced at point u, 0 by the load $P=1$ at the point x, b:

$$M=\sum_{1,2,\cdots}^{n}C_n\sin nu\sin nx\text{----------}(80)$$

where the coefficients C_n are functions of b only. The same formula, only with different values of C_n, expresses the corresponding value of M_y. The two loads $P=1$ at the points u, b and $u+a$, b then produce the moment,

$$M=\sum_{1,2,\cdots}^{n}C_n\sin nu\,(\sin nu+\sin n(u+a))$$

$$=\sum_{1,2,\cdots}^{n}\frac{C_n}{2}(1-\cos 2nu+\cos na-\cos n(2u+a))\text{-}(81)$$

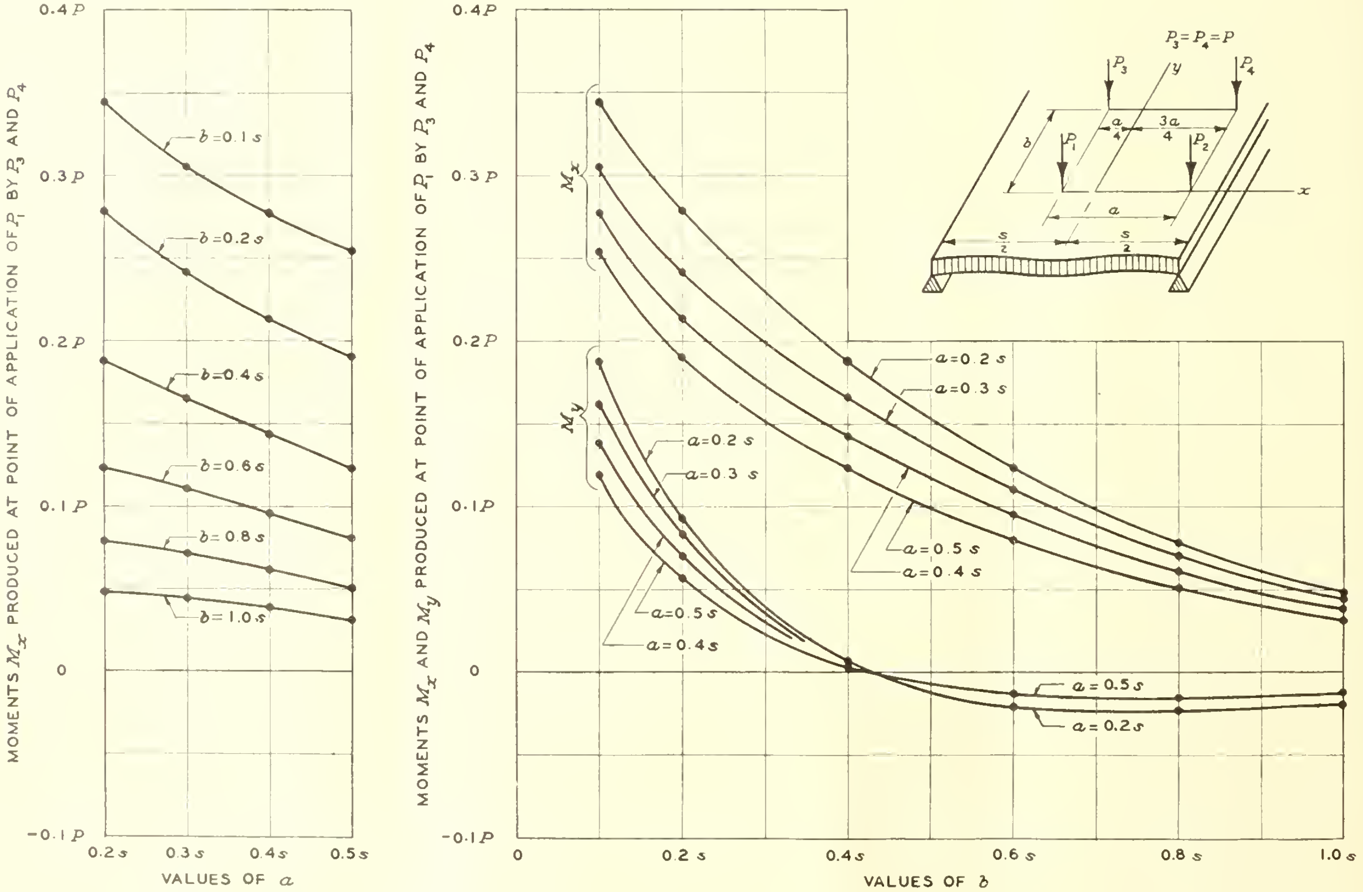


Figure 15.—Bending Moments Produced at Point of Application of P_1 by Two Loads, $P_3=P$ and $P_4=P$ (from Equation 88 and Table 6). Poisson's Ratio, $\mu=0.15$

One finds, furthermore,

$$\frac{dM}{du}=\sum_{1,2,\cdots}^{n} n\ C_n\ (\sin 2nu+\sin n(2u+a))\text{----}(82)$$

which becomes zero when $u=\frac{\pi}{2}-\frac{a}{4}$, or $v=\frac{a}{4}$. It is concluded that M_x or M_y, respectively, reaches an extreme value when $v=\frac{a}{4}$, and that this value is a maximum when M in equation 80 is positive for all values of x between 0 and π. That is, the rule by which two equal loads are placed on a beam so as to produce a maximum moment, and which was found to apply to P_1 and P_2, applies also to P_3 and P_4.

The y-axis is now moved back to the center-line of the slab, and the span is assumed to have any value, s. The two loads $P_3=P$ and $P_4=P$ are placed as shown in Figure 15. For these two loads equations 44 and 45 give equal values of A, but different values of B, which will be denoted by B_3 and B_4, respectively. One finds

$$A=\cosh\frac{\pi b}{s}+\cos\frac{\pi a}{2s}\text{--------}(83)$$

$$B_3=\cosh\frac{\pi b}{s}-1\text{--------}(84)$$

$$B_4=\cosh\frac{\pi b}{s}-\cos\frac{\pi a}{s}\text{--------}(85)$$

The moments produced by the two loads, $P_3=P$ and $P_4=P$, then may be expressed as follows, by use of equations 51 and 54:

$$\left.\begin{matrix}M_x\\M_y\end{matrix}\right\}=\frac{(1+\mu)P}{8\pi}\log_e\frac{A^2}{B_3B_4}\pm$$

$$\frac{(1-\mu)Pb}{8s}\sinh\frac{\pi b}{s}\left(\frac{1}{B_3}+\frac{1}{B_4}-\frac{2}{A}\right)\text{--------}(86)$$

$$M_{xy}=-\frac{(1-\mu)Pb}{8s}\ \frac{\sin\frac{\pi a}{s}}{B_4}\text{--------}(87)$$

or, with $\mu=0.15$,

$$\left.\begin{matrix}M_x\\M_y\end{matrix}\right\}=0.10536\,P\log_{10}\frac{A^2}{B_3B_4}\pm$$

$$0.10625\,\frac{Pb}{s}\sinh\frac{\pi b}{s}\left(\frac{1}{B_3}+\frac{1}{B_4}-\frac{2}{A}\right)\text{--------}(88)$$

$$M_{xy}=-0.10625\,\frac{Pb}{s}\ \frac{\sin\frac{\pi a}{s}}{B_4}\text{--------}(89)$$

Table 6 contains values computed from equations 88 and 89 with use of equations 83, 84, and 85. Figure 15 shows curves representing equation 88.

A comparison of equations 87 and 76 shows that the twisting moment M_{xy} produced at the point $-\frac{a}{4}$, 0, by the two loads P at the points $-\frac{a}{4}$, b and $\frac{3a}{4}$, b, is equal to the twisting moment M'_{xy} produced at the point 0, 0 by a single load P at point $\frac{a}{2}$, $\frac{b}{2}$. Figures 13 and 14, therefore, supply the necessary information about the twisting moments produced by P_3 and P_4.

COMBINED EFFECTS OF FOUR LOADS

To produce the greatest possible bending moments M_x and M_y at the point of application of P_1, the four loads, P_1, P_2, P_3, and P_4, each equal to P, are placed as shown in Figure 16. The combined effects of P_1 and P_2 are given in equations 71 and 72 and in Table 4, in conjunction with equations 60 and 61 and Table 1. The combined effects of P_3 and P_4 are defined by equations 83 to 89 and are given in Table 6. By adding the results, one finds the moments M_x, M_y, and M_{xy} produced at the point of application of P_1 by the combined action of the four loads. From these values one obtains the principal moments M_1 and M_2, that is, the greatest bending moment and the smallest bending moment at the particular point, and also the angle ψ between the x-axis and the direction of M_1, by the following formulas, which are analogous to those applying to a plane state of stresses:

$$\left.\begin{matrix}M_1\\M_2\end{matrix}\right\}=\frac{M_x+M_y}{2}\pm\sqrt{\frac{(M_x-M_y)^2}{4}+M^2{}_{xy}}\text{----}(90)$$

$$\tan 2\psi=\frac{2M_{xy}}{M_x-M_y}\text{--------}(91)$$

Table 6 contains values, for $P=1$, of M_{xy} and of the amounts M_x-M_{0x}, M_y-M_{0x}, M_1-M_{0x}, and M_2-M_{0x} which are to be added to M_{0x} (as given by equation 61 and in Table 1) in order to obtain the moments due to the four loads. The curves in Figures 16 and 17 show the values of M_1-M_{0x}, M_2-M_{0x}, and ψ for different values of a and b.

An examination of Figure 17 shows that the following formula applies as a crude approximation, giving values which are not too small, when $0.3s<a<0.5s$, and $0.3s<b<s$:

$$\frac{M_1-M_{0x}}{P}=\frac{0.4s}{2a+b}-0.14\text{--------}(92)$$

Using this formula in conjunction with the roughly approximate formula, equation 66, one finds

$$M_1=\frac{Ps}{2.32s+8c}+\frac{0.4Ps}{2a+b}-0.14P\text{----}(93)$$

DETERMINATION OF CHANGES CAUSED BY INTRODUCTION OF BEAMS IN DIRECTION OF x

Let the slab, extending indefinitely far in the directions of $+y$ and $-y$, be loaded by a force P at the point x, y_1 and by a force $-P$ (that is, an upward force P) at the point x, $2b_1-y_1$ (where $b_1>y_1$). The deflections, z, and bending moments, M_x and M_y, produced by the two loads at the line $y=b_1$ will neutralize each other, so that at this line one finds $z=\Delta z=0$. The part of the slab for which $y<b_1$, therefore, behaves as if the slab had a simply supported edge at $y=b_1$. Likewise,[18] if one introduces a set of loads $+P$ at the points $x=x_1$, $y=y_1+2nl$, and loads $-P$ at the points $x=x_1$, $y=2b_1-y_1+2nl$, with $n=0$, ±1, ±2, . . ., the part of the slab between the lines $y=b_1$ and $y=b_1-l$ will act as a rectangular slab which has simply supported edges

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[18] A. Nádai, Die elastischen Platten, 1925, p. 84.

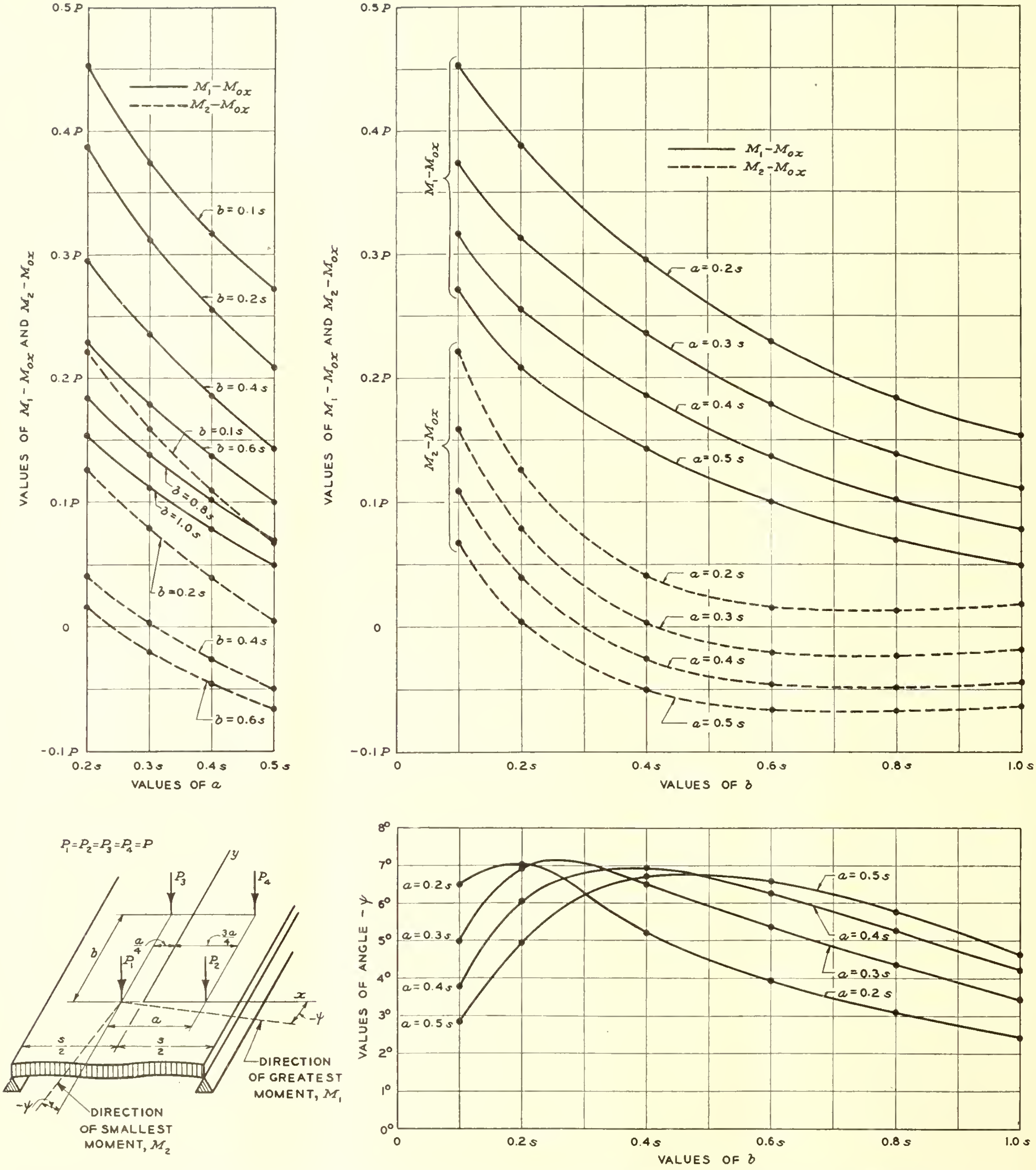

Figure 16.—Combined Effects of Four Loads. Amounts to be Added to M_{0x} to Obtain Principal Moments M_1 and M_2, Produced at Point of Application of P_1 by Joint Action of Four Loads, P_1, P_2, P_3, and P_4, Each Equal to P. Angles Between x-Axis and M_1. (From Equations 60, 72, and 83 to 91, and Table 6.) Poisson's Ratio, $\mu=0.15$

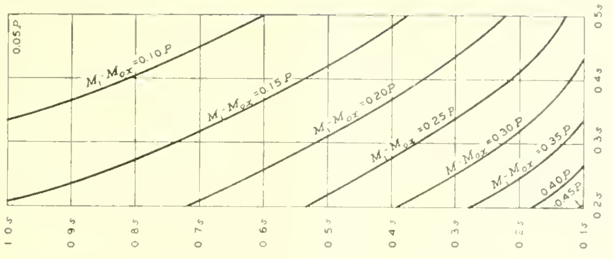

FIGURE 17.—COMBINED EFFECTS OF FOUR LOADS. CURVES FOR CONSTANT VALUES OF M_1-M_{0x}, DETERMINED FROM FIGURE 16. POISSON'S RATIO, $\mu=0.15$.

TABLE 6.—*Bending moments M_x and M_y produced at point $-\frac{a}{4}$, 0 by two loads, $P=1$, at points $-\frac{a}{4}$, b and $\frac{3a}{4}$, b, computed from equations 83, 84, 85, and 88, and represented graphically in Figure 15. Twisting moments M_{xy} produced at the same point by the same two loads, or by these loads in conjunction with P_1 and P_2, computed from equations 85 and 89. Amounts M_x-M_{0x}, M_y-M_{0x}, M_1-M_{0x}, and M_2-M_{0x} to be added to M_{0x} to obtain the moments produced at point $-\frac{a}{4}$, 0 by the combined action of four loads, $P=1$, applied as shown in Figure 16, M_1 and M_2 being the principal moments at the point. Angle ψ between x-axis and direction of M_1. The values of M_1-M_{0x}, M_2-M_{0x}, and ψ are computed from equations 60, 72, and 83 to 91, and are represented graphically in Figures 16 and 17. Poisson's ratio, $\mu=0.15$*

		From two loads			From four loads					
$\frac{a}{s}$	$\frac{b}{s}$	M_x	M_y	$-M_{xy}$	M_x-M_{0x}	M_y-M_{0x}	M_1-M_{0x}	M_2-M_{0x}	$-\psi$	
									°	′
0.2	0.1	0.34483	0.18703	0.02594	0.45003	0.22459	0.45297	0.22165	6	29
	.2	.27823	.09281	.03163	.38343	.13037	.38733	.12647	7	1
	.4	.18788	.00566	.02292	.29308	.04322	.29517	.04113	5	12
	.6	.12318	−.02030	.01464	.22838	.01726	.22939	.01625	3	57
	.8	.07838	−.02348	.00925	.18358	.01408	.18408	.01358	3	7
	1.0	.04864	−.01934	.00579	.15384	.01822	.15408	.01798	2	26
.3	.1	.30536	.16128	.01861	.37250	.16078	.37412	.15916	4	59
	.2	.24195	.08321	.02790	.30909	.08271	.31248	.07932	6	55
	.4	.16583	.00691	.02622	.23297	.00641	.23597	.00341	6	31
	.6	.11039	−.01767	.01854	.17753	−.01817	.17927	−.01991	5	22
	.8	.07074	−.02106	.01222	.13788	−.02156	.13881	−.02249	4	22
	1.0	.04405	−.01747	.00781	.11119	−.01797	.11166	−.01844	3	27
.4	.1	.27690	.13864	.01364	.31634	.11044	.31724	.10954	3	46
	.2	.21347	.07023	.02258	.25291	.04203	.25530	.03964	6	3
	.4	.14361	.00603	.02542	.18305	−.02217	.18615	−.02527	6	57
	.6	.09563	−.01521	.01981	.13507	−.04341	.13724	−.04558	6	16
	.8	.06138	−.01824	.01369	.10082	−.04644	.10208	−.04770	5	16
	1.0	.03825	−.01517	.00896	.07769	−.04337	.07835	−.04403	4	13
.5	.1	.25423	.11909	.01012	.27146	.06868	.27196	.06818	2	51
	.2	.19030	.05674	.01765	.20753	.00633	.20907	.00479	4	59
	.4	.12297	.00339	.02238	.14020	−.04702	.14284	−.04966	6	43
	.6	.08037	−.01327	.01892	.09760	−.06368	.09979	−.06587	6	36
	.8	.05113	−.01535	.01368	.06836	−.06576	.06974	−.06714	5	46
	1.0	.03171	−.01261	.00917	.04894	−.06302	.04969	−.06377	4	39

at $x=\pm\frac{s}{2}$ and at $y=b_1$ and $y=b_1-l$, and which is loaded by the force P at the point x_1, y_1. This equivalence of two cases leads to a simple determination of the action of the rectangular slab by use of the results found for the slab extending infinitely far in the directions of x and y.

As the first example, consider a slab which has simply supported edges at $x=\pm\frac{s}{2}$ and at $y=\frac{s}{2}$, and which extends infinitely far in the direction of $-y$. Let this slab be loaded by a force P at the point 0, 0. The slab extending infinitely far also in the direction of $+y$ then is to be loaded by the additional force $-P$ at the point $x=0$, $y=2b_1-y_1=s$. Values stated in the first section of Table 5, then give at the point 0, 0:

$$M_x=M_{0x}-0.0263P$$

$$M_y=M_{0y}+0.0105P$$

As a second example, consider a square slab loaded at the center. With $b_1=\frac{s}{2}$, $l=s$, the loads P are introduced at the points $x=0$, $y=0$, $\pm 2s$, $\pm 4s$, . . . and the loads $-P$ are introduced at the points $x=0$, $y=\pm s$, $\pm 3s$, ... Then one finds at point 0, 0, by use of Table 5 and equation 62:

$$\begin{aligned} M_x &= M_{0x}+2(-0.0263+0.019-0.0001+.\ .\ .)P \\ &= M_{0x}-0.0490P, \\ M_y &= (M_{0x}-0.0676P)+2(0.0105-0.0013+ \\ &\quad 0.0001-.\ .\ .)P \\ &= M_{0x}-0.0490P. \end{aligned}$$

The equality of the two moments, so determined, is noted. They should be equal since the slab is square.

EFFECTS OF CHANGING FROM SIMPLY SUPPORTED EDGES TO FIXED EDGES INVESTIGATED

A rectangular slab is considered which has simply supported edges at $x=\pm\frac{s}{2}$ and $y=\pm\frac{l}{2}$, and is loaded by a single force P at the center, point 0, 0. By introducing the symbols,

$$\left.\begin{aligned} &\omega_n=\frac{n\pi}{l},\ \alpha_n=\frac{\omega_n s}{2}=\frac{n\pi s}{2l}, \\ &\text{where } n=1,\ 3,\ 5,\cdot\cdot\cdot \end{aligned}\right\}\text{-----------}(94)$$

one may show that the following formula expresses the deflection of this slab at the point x, y when $x\geqq 0$:

$$z=\frac{Pl^2}{2\pi^3 N}\sum_{1,3,5,\cdot\cdot}^{n}\frac{\cos\omega_n y}{n^3}\left[\left(\tanh\alpha_n-\frac{\alpha_n}{\cosh^2\alpha_n}\right)\cosh\omega_n x\right.$$
$$\left.-\omega_n x\tanh\alpha_n\sinh\omega_n x-\sinh\omega_n x+\omega_n x\cosh\omega_n x\right]\text{-}(95)$$

To verify this formula, one may begin by observing that $z=0$ when $x=\frac{s}{2}$ (giving $\omega_n x=\alpha_n$) and when $y=\pm\frac{l}{2}$. One finds

$$\frac{\partial z}{\partial x}=\frac{Pl}{2\pi^2 N}\sum_{1,3,5,\cdot\cdot}^{n}\frac{\cos\omega_n y}{n^2}\left[\omega_n x\sinh\omega_n x\right.$$
$$\left.-\frac{\alpha_n}{\cosh^2\alpha_n}\sinh\omega_n x-\omega_n x\tanh\alpha_n\cosh\omega_n x\right]\text{---}(96)$$

which becomes zero when $x=0$. By further differentiations one finds

$$\Delta z=\frac{P}{\pi N}\sum_{1,3,5,\cdot\cdot}^{n}\frac{\cos\omega_n y}{n}\left[-\tanh\alpha_n\cosh\omega_n x+\sinh\omega_n x\right]\text{-}(97)$$

which becomes zero when $x=\frac{s}{2}$ or $y=\pm\frac{l}{2}$. The vertical shear in a section parallel to the y-axis becomes, according to equation 17,

$$V_x=-N\frac{\partial\Delta z}{\partial x}=-\frac{P}{l}\sum_{1,3,5,\cdots}^{n}\cos\omega_n y\left[\cosh\omega_n x-\tanh\alpha_n\sinh\omega_n x\right]\qquad(98)$$

When $x=0$, this series assumes the divergent form,

$$V_x=-\frac{P}{l}\sum_{1,3,5,\cdots}^{n}\cos\frac{n\pi y}{l}\qquad(99)$$

By comparing this equation with the expression for V_y in equation 29, it is seen that equation 99 expresses the fact that the boundary condition in the section $x=0$, resulting from the presence of the concentrated load P at point 0, 0 is satisfied. By further differentiations of equation 97 one finds $\Delta^2 z=0$. Thus, the function z in equation 95 satisfies the equation of flexure as well as all the conditions of the boundary.

The slope at the edge $x=\frac{s}{2}$ is of particular interest. Equation 96 gives at this line

$$\frac{\partial z}{\partial x}=-\frac{Pl}{2\pi^2 N}\sum_{1,3,5,\cdots}^{n}\frac{\cos\omega_n y}{n^2}\frac{\alpha_n\sinh\alpha_n}{\cosh^2\alpha_n}\qquad(100)$$

Consider now the function,

$$z_1=-\frac{Pl^2}{\pi^3 N}\sum_{1,3,5,\cdots}^{n}\frac{\cos\omega_n y}{n^3}\frac{\alpha_n\tanh\alpha_n}{\sinh 2\alpha_n+2\alpha_n}\left[\alpha_n\tanh\alpha_n\cosh\omega_n x-\omega_n x\sinh\omega_n x\right]\qquad(101)$$

This function is found to have the following properties:

At $y=\pm\frac{l}{2}$: $z_1=\Delta z_1=0$.

At $x=0$: $\frac{\partial z_1}{\partial x}=0$, $V_x=-N\frac{\partial\Delta z_1}{\partial x}=0$.

At $x=\frac{s}{2}$: $z_1=0$, $\frac{\partial z_1}{\partial x}=-\frac{\partial z}{\partial x}$ (equation 100).

At all points: $\Delta^2 z_1=0$.

It follows that the function $z'=z+z_1$ represents the deflection (for $x>0$) of a rectangular slab which has simply supported edges at $y=\pm\frac{l}{2}$ and fixed edges at $x=\pm\frac{s}{2}$, and which is loaded by the force P at the point 0, 0. That is, z_1 represents the change of deflection caused by fixing the two edges parallel to the y-axis. The corresponding changes of the moments in the section $x=0$ are then expressed as follows, by use of equations 11 and 12:

$$M'_x-M_x=N\left[-\frac{\partial^2 z_1}{\partial x^2}-\mu\frac{\partial^2 z_1}{\partial y^2}\right]_{x=0}$$

$$=\frac{Ps}{2l}\sum_{1,3,5,\cdots}^{n}\frac{\cos\omega_n y\tanh\alpha_n}{\sinh 2\alpha_n+2\alpha_n}\left[(1-\mu)\alpha_n\tanh\alpha_n-2\right]\qquad(102)$$

$$M'_y-M_y=N\left[-\frac{\partial^2 z_1}{\partial y^2}-\mu\frac{\partial^2 z_1}{\partial x^2}\right]_{x=0}$$

$$=\frac{Ps}{2l}\sum_{1,3,5,\cdots}^{n}\frac{\cos\omega_n y\tanh\alpha_n}{\sinh 2\alpha_n+2\alpha_n}\left[-(1-\mu)\alpha_n\tanh\alpha_n-2\mu\right]\qquad(103)$$

The values stated in Table 7 have been computed from equations 102 and 103 with $\mu=0.15$ and $l=2.5\pi s=7.854s$. The value of l is so large that changing it to infinity would make no noticeable difference. The results are represented graphically by the curves in the lower part of Figure 9. The curves for M'_x and M'_y in the upper part of Figure 9 were constructed from the curves for M_x and M_y by laying off intercepts equal to M'_x-M_x and M'_y-M_y.

From the values given in the table for point $x=y=0$, one finds by use of equation 60:

$$M'_{0x}=M_{0x}-0.0699P\qquad(104)$$

$$M'_{0y}=M_{0x}-0.06764P-0.03863P=M_{0x}-0.1063P\qquad(105)$$

These formulas explain the two scales farthest to the right in Figure 6.

TABLE 7.—*Changes, M'_x-M_x and M'_y-M_y, of the bending moments at the center-line of the slab, caused by change from simply supported edges to fixed edges, when the slab is loaded by the force $P=1$ at point 0, 0. Values computed from equations 102 and 103, and shown graphically in Figure 9. Poisson's ratio, $\mu=0.15$*

$\frac{y}{s}$	M'_x-M_x	M'_y-M_y	$\frac{y}{s}$	M'_x-M_x	M'_y-M_y
0	−0.06994	−0.03863	1.0	−0.0248	0.0059
.2	−.06755	−.0323	1.5	−.00764	.00386
.4	−.0602	−.0181	2	−.00198	.00129
.6	−.0489	−.0048			

SLAB CANTILEVERED FROM A SINGLE FIXED EDGE INVESTIGATED

The slab shown in Figure 18 has a fixed edge along the y-axis, and is assumed to cover one-half of the xy-plane, the part for which x is positive. Consider the bending moment M_x produced at the point 0, 0 by a load $P=1$ at the point x, y. The locus of a point with the three rectangular coordinates x, y, M_x is the influence surface for M_x. It is well known that any influence diagram may be obtained as a deflection diagram by introducing the proper discontinuity at the point under investigation. In applying this principle to the present case, one is to determine a surface with coordinates x, y, z, so that the function z satisfies the following conditions: It is required, first, that the equation of flexure,

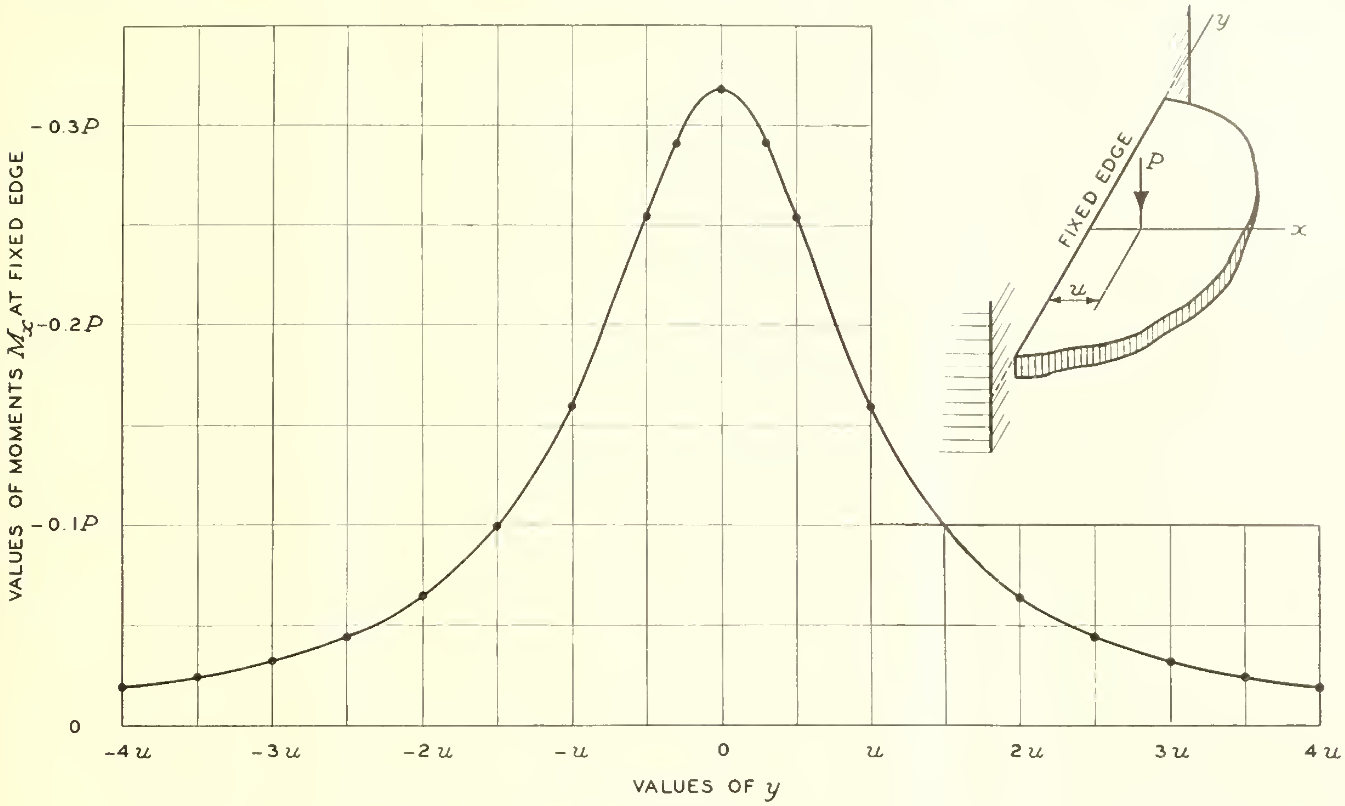

FIGURE 18.—BENDING MOMENTS AT FIXED EDGE OF LARGE SLAB (FROM EQUATION 110 AND TABLE 8)

$\Delta^2 z=0$, be satisfied at all points except at the point 0, 0, where the function has a singularity; secondly, that $z=\frac{\partial z}{\partial x}=0$ at the edge $x=0$, except at the point 0, 0; thirdly, that z and Δz shall converge toward zero when x or y increases indefinitely; and fourthly, that the singularity at the point 0, 0 shall represent a proper concentration of slope at the particular point.

One may think of this concentration of a slope as one thinks of the concentration of a force: The distributed force $p=p(y)$ represents a total load $P=\int p dy$; by changing the function p gradually, but maintaining the value of the integral, the distributed force may be changed into the concentrated load. The function $\frac{\partial z}{\partial x}$ may be concentrated by gradual change in the same manner as the function p.

A function z of the following form is found to satisfy the requirements:

$$z=\frac{kx^2}{x^2+y^2} \text{ - - - - - - - - - - - - - -(106)}$$

where k is a constant. A simple method of determining this constant is by noting that a distributed load, one unit per unit of length, on the line $x=1$, produces a moment $M_x=-1$ at the edge. That is,

$$-1=\int_{-\infty}^{\infty}\frac{kdy}{1+y^2}=k\pi,\text{ or, } k=-\frac{1}{\pi} \text{ - - - - -(107)}$$

Since z in equation 106 is interpreted as equal to the desired moment M_x at the point 0, 0, one finds

$$M_x=-\frac{x^2}{\pi(x^2+y^2)} \text{ - - - - - - - - - - - - -(108)}$$

or, in terms of the angle θ from the x-axis to the radius vector,

$$M_x=-\frac{1}{\pi}\cos^2\theta \text{ - - - - - - - - - - - -(109)}$$

The result expressed in equation 108 may be restated as follows: A load P at the point u, 0 produces a moment diagram at the edge with the equation,

$$M_x=-\frac{P}{\pi}\frac{1}{1+\frac{y^2}{u^2}} \text{ - - - - - - - - - - -(110)}$$

Table 8 and Figure 18 show values computed from this equation.

TABLE 8.—*Moments M_x at fixed edge in Figure 18 when P=1, computed from equation 110*

$\frac{y}{u}$	$-M_x$	$\frac{y}{u}$	$-M_x$
0	0.3183	2.0	0.0637
.3	.2920	2.5	.0439
.5	.2546	3.0	.0318
1.0	.1592	3.5	.0240
1.5	.0979	4.0	.0187

It is of some interest to know the bending moment M_x produced at point 0, 0 in Figure 18 when the load P is distributed uniformly over a circle with diameter c tangent to the edge at point 0, 0. Equation 109 gives

$$M_x=-\int_{-\frac{\pi}{2}}^{\frac{\pi}{2}}d\theta\int_0^{c\cos\theta}\frac{4Prdr}{\pi c^2}\frac{\cos^2\theta}{\pi},$$

or,

$$M_x=-\frac{3P}{4\pi} \text{ -(111)}$$

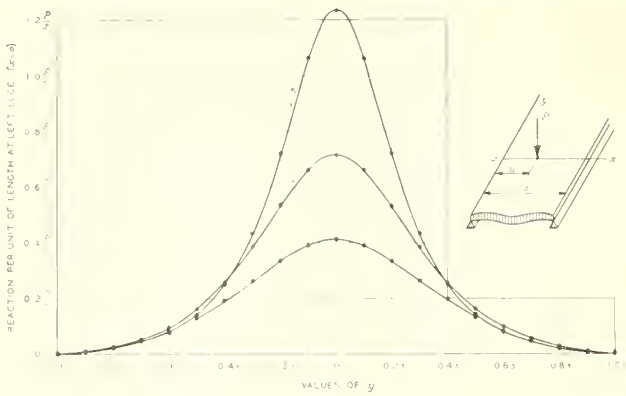

Figure 19.—Reactions at Left Edge (from Equation 115 and Table 9). Poisson's Ratio, $\mu=0.15$

This moment is three-fourths of the moment produced when the load P is concentrated at the center of the circle.

REACTIONS DETERMINED

Consider the case shown in Figure 19. The y-axis is at the left edge. The two edges are simply supported, and the slab extends infinitely far in the directions of $+y$ and $-y$. The load P is applied at the point u, 0. From equations 17, 43 to 46, and 52, and by consideration of the changed position of the y-axis, one finds the shear V_x and the twisting moment M_{xy} at the left edge,

$$V_x=\frac{P}{2s}\,\frac{\sin\frac{\pi u}{s}}{\cosh\frac{\pi y}{s}-\cos\frac{\pi u}{s}}\quad\text{(112)}$$

$$M_{xy}=\frac{(1-\mu)Py}{4s}\,\frac{\sin\frac{\pi u}{s}}{\cosh\frac{\pi y}{s}-\cos\frac{\pi u}{s}}\quad\text{(113)}$$

According to equation 19 this combination of shears and twisting moments is equivalent to the vertical reaction,

$$R_x=V_x+\frac{\partial M_{xy}}{\partial y}=\frac{(3-\mu)P}{4s}\,\frac{\sin\frac{\pi u}{s}}{\cosh\frac{\pi y}{s}-\cos\frac{\pi u}{s}}\times\left(1-\frac{1-\mu}{3-\mu}\,\frac{\frac{\pi y}{s}\sinh\frac{\pi y}{s}}{\cosh\frac{\pi y}{s}-\cos\frac{\pi u}{s}}\right)\quad\text{(114)}$$

or, with $\mu=0.15$,

$$R_x=0.7125\frac{P}{s}\,\frac{\sin\frac{\pi u}{s}}{\cosh\frac{\pi y}{s}-\cos\frac{\pi u}{s}}\times\left(1-0.29825\,\frac{\frac{\pi y}{s}\sinh\frac{\pi y}{s}}{\cosh\frac{\pi y}{s}-\cos\frac{\pi u}{s}}\right)\quad\text{(115)}$$

Table 9 and Figure 19 show values computed from equation 115.

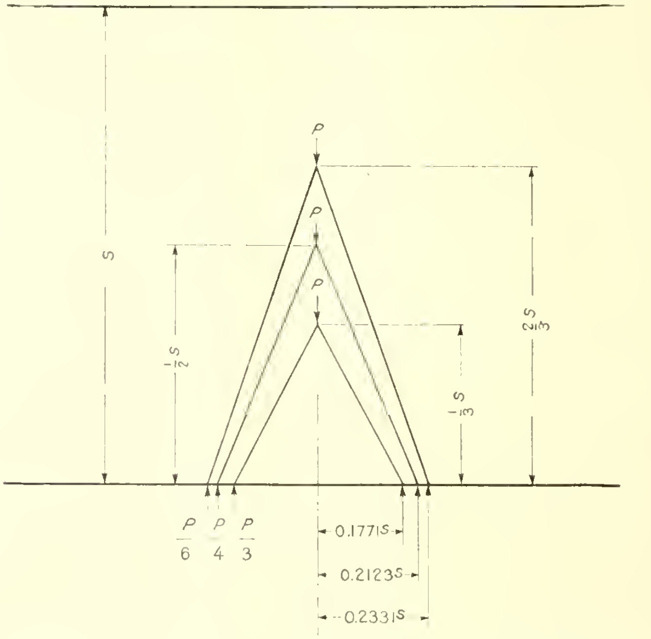

Figure 20.—Positions of Resultants, Each Representing Left Half or Right Half of the Diagram of Reactions, in Cases Shown in Figure 19 (from Equation 116). Poisson's Ratio, $\mu=0.15$

Table 9.—*Reactions R_x produced at the left edge by a load $P=1$ on the x-axis at the distance u from the edge, computed from equation 115, and represented graphically in Figure 19. Poisson's ratio, $\mu=0.15$*

$\frac{y}{s}$	Reaction R_x			$\frac{y}{s}$	Reaction R_x		
	$u=\frac{1}{3}s$	$u=\frac{1}{2}s$	$u=\frac{2}{3}s$		$u=\frac{1}{3}s$	$u=\frac{1}{2}s$	$u=\frac{2}{3}s$
0	1.2340	0.7125	0.4113	0.7	0.0427	0.056	0.0515
.1	1.0612	.659	.3904	.8	.0211	.030	.0290
.2	.7200	.530	.3354	.9	.0085	.014	.0144
.3	.4336	.382	.2636	1.0	.0014	.004	.0054
.4	.2503	.256	.1923	1.07	----------	.000	----------
.5	.1423	.162	.1315	1.1	----------	−.001	----------
.6	.0795	.098	.0849				

For the purpose of computing bending moments in the supporting beams, it is of interest to know the position of the resultant force representing the right half of each of the symmetrical diagrams in Figure 19. The distance, y_R from the point 0, 0 to this resultant is defined by the equation of moments,

$$y_R\int_0^\infty R_x dy=\int_0^\infty R_x y dy\quad\text{(116)}$$

The integral on the left side of this equation becomes in the three cases $\frac{P}{3}$, $\frac{P}{4}$, and $\frac{P}{6}$, respectively. The integral on the right side was determined in each of the three cases by numerical integration. By this method the three distances y_R shown in Figure 20 were obtained. One may interpret these results by saying that the resultant of the whole reaction is resolved in each case into two sub-resultants, each representing one-half of the diagram, and located as shown in Figure 20. An examination of the values given in Figure 20 shows that the following formula applies as a rough approximation:

$$y_R=0.3\sqrt{us}\quad\text{(117)}$$

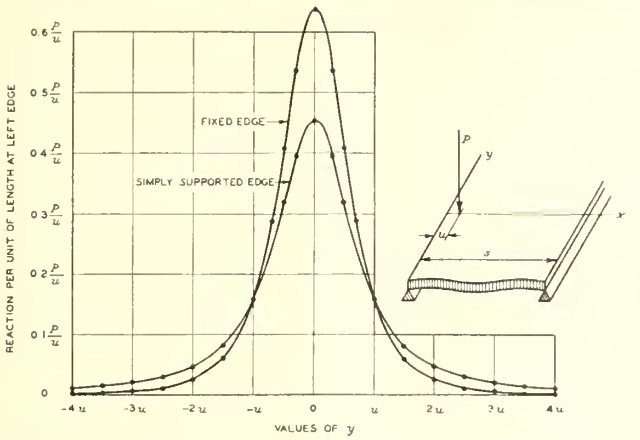

Figure 21.—Reactions Produced by Load Close to Edge (from Equations 119 and 120, and Table 10) In Case of Simply Supported Edge, Poisson's Ratio, $\mu=0.15$

When the distance u from the edge to the load becomes small in comparison with the span, one may simplify equation 114 by substituting,

$$\sin\frac{\pi u}{s}=\frac{\pi u}{s},\ \sinh\frac{\pi y}{s}=\frac{\pi y}{s},\ \cosh\frac{\pi y}{s}-\cos\frac{\pi u}{s}=\frac{\pi^2}{2s^2}(y^2+u^2).$$

Then one finds

$$R_x=\frac{P}{2\pi}\frac{u}{u^2+y^2}\left(1+\mu+2(1-\mu)\frac{u^2}{u^2+y^2}\right)\text{- - -}(118)$$

or, with $\mu=0.15$,

$$R_x=0.1830\frac{P}{u}\frac{1}{1+\frac{y^2}{u^2}}\left(1+\frac{1.4783}{1+\frac{y^2}{u^2}}\right)\text{- - -}(119)$$

When the edge is fixed, one finds, by a procedure similar to that which led to equation 110,

$$R_x=\frac{2P}{\pi u}\frac{1}{\left(1+\frac{y^2}{u^2}\right)^2}=\frac{P}{u}\frac{0.6366}{\left(1+\frac{y^2}{u^2}\right)^2}\text{- - - - - - - - - - -}(120)$$

Table 10 and Figure 21 show results computed from equations 119 and 120.

If x is substituted for u in equations 118 to 120, these equations may be interpreted as defining the reaction R_x produced at point 0, 0 by a load P at point x,y. In terms of polar coordinates, with $x=r\cos\theta$, $y=r\sin\theta$, one finds then at point 0, 0 at the simply supported edge

$$R_x=\frac{P\cos\theta}{2\pi r}(1+\mu+2(1-\mu)\cos^2\theta)\text{- - - -}(121)$$

Table 10.—*Reactions R_x produced at the left edge by a load $P=1$ on the x-axis at a small distance u from the edge, computed from equations 119 and 120, and represented graphically in Figure 21. In case of a simply supported edge, Poisson's ratio, $\mu=0.15$*

$\frac{y}{u}$	Values of R_x		$\frac{y}{u}$	Values of R_x	
	Simply supported edge	Fixed edge		Simply supported edge	Fixed edge
0	0.4535	0.6366	2.0	0.0474	0.0255
.3	.3956	.5358	2.5	.0304	.0121
.5	.3195	.4074	3.0	.0210	.0064
.7	----------	.2867	3.5	.0154	.0036
1.0	.1591	.1592	4.0	.0117	.0022
1.5	.0819	.0603			

and at point 0, 0 of the fixed edge

$$R_x=\frac{2P}{\pi r}\cos^3\theta\text{- - - - - - - - - - -}(122)$$

One may use these formulas to determine the reaction per unit of length produced at point 0, 0 when the load P is distributed uniformly over the area of a small circle with diameter c, tangent to the edge at point 0, 0. By integrating over the area in the same manner as in deriving equation 111, one finds at point 0, 0 of the simply supported edge

$$R_x=\frac{5-\mu}{2}\frac{P}{\pi c}\text{- - - - - - - - - - - -}(123)$$

and at point 0, 0 of the fixed edge

$$R_x=\frac{3P}{\pi c}\text{- - - - - - - - - -}(124)$$

It is noted that $\frac{P}{\pi c}$ is the value that would be obtained if the force were distributed uniformly over the length of the circumference of the circle. At the fixed edge the twisting moments are zero, and R_x is the same as the shear V_x. At the simply supported edge, on the other hand, the presence of the twisting moments cause R_x in equation 123 to be larger than the shear V_x at the same point. One finds at point 0, 0

$$V_x=\frac{2P}{\pi c}\text{- - - - - - - - - -}(125)$$

That is, the shear V_x at point 0, 0 is twice the value that would be found by distributing the load uniformly over the circumference of the circle.

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ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

FEBRUARY 28, 1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	2.103.3	$ 2.463.825.46	$ 1,215,498.79	87.0	21.0	108.0						$ 3.448.250.28	Alabama
Arizona	810.3	3.000.859.26	2,245,723.42	144.5	121.2	265.7	$ 705,675.44	$ 531.303.02	0.3	14.9	15.2	2,177.991.65	Arizona
Arkansas	1.764.5	3.354,511.57	1,601.744.38	142.0	28.7	170.7	392,930.31	169.199.40	19.2	8.2	27.4	2.593.119.20	Arkansas
California	1.947.9	6.532.917.52	2,900,406.90	163.5	9.2	172.7	2,734.581.03	702,156.06	37.1	15.0	53.1	1,697,621.10	California
Colorado	1.184.6	4.295.287.20	2.278.728.55	168.9	30.3	199.2	759,039.51	371,406.23	32.7	8.7	41.4	2.222.317.24	Colorado
Connecticut	240.3	1.095.990.67	455.428.90	6.5		6.5	673,564.46	272,075.09	3.6		3.6	914,520.46	Connecticut
Delaware	249.3	601,315.95	222.917.99	16.5		16.5	280,955.00	125.521.00	9.4		9.4	286,085.39	Delaware
Florida	467.2	5,032,471.20	2.284.205.05	102.1	5.5	107.5	601,441.91	247.779.63	16.9		16.9	1.322,041.00	Florida
Georgia	2.721.1	1.131.914.96	536.760.13	38.2	8.5	46.7						4.004.075.65	Georgia
Idaho	1,197.6	705.480.50	426.310.70	55.7	24.1	79.8	67,760.98	41.076.63	8.6	3.7	12.3	1.420.447.00	Idaho
Illinois	2,034.5	15.137.362.22	6.576.755.95	430.1		430.1	529,000.00	264.000.00	18.3		18.3	5,777.748.00	Illinois
Indiana	1.344.0	7,509.467.02	3,525.586.95	233.1		233.1	374.724.44	178,170.00	11.9		11.9	1.944.445.43	Indiana
Iowa	3.130.6	818.396.20	343.538.53	15.1	18.5	33.6	4.594,049.99	2.034.098.01	30.7	138.8	169.5	35,944.84	Iowa
Kansas	2.685.5	5.996,177.15	2.467.043.79	328.1	15.7	343.8	872,186.00	415.836.58	34.0	11.0	45.0	1,949,575.68	Kansas
Kentucky	1.397.9	4,238,953.63	2.013.037.35	235.5	20.2	255.7						1.846,732.84	Kentucky
Louisiana	1.357.0	3.767,314.15	1,842,730.53	109.1	11.2	120.3	458.872.51	224.703.86	17.9	1.3	19.1	1,701.842.24	Louisiana
Maine	520.0	1.905.021.13	736,937.56	47.5		47.5	87.912.01	43,956.00	5.5		5.5	1,523.961.32	Maine
Maryland	630.7	1.428.486.50	598,781.99	58.0		58.0	130.449.50	65.224.75	5.9		5.9	569.879.16	Maryland
Massachusetts	655.7	2.119.194.97	658.992.52	26.1		26.1	694.763.28	209.735.99	14.6		14.6	2,219.518.11	Massachusetts
Michigan	1,615.9	9,046.446.10	3.892.560.50	209.5	19.6	229.1	865.000.00	284,258.31	17.8	10.9	28.7	2.124.005.15	Michigan
Minnesota	4.107.5	4,778,319.69	1.384.300.00	123.5	85.3	208.8	4.741,128.77	973.165.90	77.3	132.9	210.2	515.066.87	Minnesota
Mississippi	1.806.0	1,965.437.01	776,543.31	70.4	7.7	78.1						2.645.070.13	Mississippi
Missouri	2.401.9	6.308.907.59	2.432.831.76	96.4	70.7	167.1	5.703,470.44	1.853.748.42	82.8	56.7	139.5	764.117.91	Missouri
Montana	1.722.2	7.241.717.11	4,217.128.99	487.0	12.9	499.9	1,106.093.36	604.529.91	86.9	36.3	123.1	3,052.191.82	Montana
Nebraska	3.589.1	6,790.134.98	3.290,038.01	330.5	155.3	485.9	396,762.02	183,657.74	19.9	15.1	35.0	2.262.918.67	Nebraska
Nevada	1.141.7	1.151.431.44	1.013,967.88	120.4	79.2	199.6	199.637.41	177.377.82		26.6	26.6	760.317.95	Nevada
New Hampshire	350.6	414,924.02	111.495.00	5.4	2.1	7.5						449.153.36	New Hampshire
New Jersey	500.2	4.136.550.77	852.015.00	56.8		56.8						1.091.304.08	New Jersey
New Mexico	1.935.0	2.907.004.00	1.842,580.28	152.9	2.1	155.0	765.700.57	447.923.65	44.4	4.4	48.8	1.032.152.51	New Mexico
New York	2.458.3	17,932,217.76	3.661.055.00	244.8		244.8	5.563,700.00	994,500.00	66.4		66.4	6,428.902.56	New York
North Carolina	1,765.2	1.297.923.70	638.411.59	74.9	22.3	97.2	1.511.206.23	755.103.11	84.3		84.3	2.213,056.34	North Carolina
North Dakota	4.176.4	1.196.709.99	482.732.47	205.4	101.7	307.1	1.630.915.56	834.183.23	313.1	110.8	423.9	1,192.762.15	North Dakota
Ohio	2.170.7	12.587.959.21	3.725.535.17	208.2	18.3	226.5	6.136.324.21	1,869,648.41	98.0	25.6	123.6	2,347.040.36	Ohio
Oklahoma	1.849.0	3.134.520.06	1.419,328.91	104.3	29.9	134.2	1.075,101.20	459.795.00	16.1	35.8	51.9	1.312.499.09	Oklahoma
Oregon	1.147.7	2,647.469.16	1,587,165.00	176.9	56.2	233.1	1.460,053.10	884.609.48	36.0	19.4	55.4	736.650.76	Oregon
Pennsylvania	2.261.3	13.931.265.99	3,341,449.44	205.6	14.1	219.7	4,410.677.44	1.254,915.30	72.7		72.7	2,131.543.64	Pennsylvania
Rhode Island	184.8	1,381.818.95	395.698.58	18.5		18.5						617,409.52	Rhode Island
South Carolina	1.895.0	3.036.983.51	1.054.207.62	86.6	24.1	110.7	222.628.47	57.000.00	.2	6.8	7.0	1.335.995.59	South Carolina
South Dakota	3,426.4	3.704.898.94	2,004,341.05	417.2	127.6	544.8	291.479.62	160.313.76	46.6	27.2	73.9	1.044.848.43	South Dakota
Tennessee	1,243.3	1.708.518.59	735,475.92	63.5		63.5	417.314.69	208.657.33	11.3	12.5	23.8	2,355.065.74	Tennessee
Texas	6,635.9	12.960.099.01	5.375.011.99	385.6	121.3	506.9	2.243.869.71	880.611.57	103.7	15.0	118.7	3.908.992.08	Texas
Utah	988.9	605.195.18	393.789.85	25.8		25.9	437.737.07	320,581.68	21.8	8.1	29.9	721.066.76	Utah
Vermont	257.8	524.915.88	209,489.96	9.3		9.3	897.266.54	202,682.46	20.1		20.1	228.425.02	Vermont
Virginia	1,413.8	2.663.948.90	1.252,004.96	120.6	15.6	136.2	1.319,405.92	587,980.18	39.6	7.7	47.3	721,194.21	Virginia
Washington	903.9	3.437.096.44	1.321,000.00	75.5	40.2	115.7	54,575.94	29,300.00	2.0		2.0	1.331.467.47	Washington
West Virginia	707.3	3.376,854.63	1,294,271.77	81.0	12.5	93.5	328.379.17	159.090.53	7.4	6.1	13.5	647,149.16	West Virginia
Wisconsin	2.280.1	4.268,427.21	1,907,945.11	153.9	3.9	157.8						1.655.997.08	Wisconsin
Wyoming	1.757.1	1,333.730.81	862,873.64	118.6	24.3	142.9						950,312.95	Wyoming
Hawaii	39.5	402,261.10	137,426.82	6.6		6.6	562.688.38	247.259.61	16.6		16.6	1,438.289.16	Hawaii
TOTALS	83.074.3	207,908,134.89	84.643.705.46	6.843.1	1.361.0	8,204.1	56.298.021.89	20,326,935.75	1.551.4	760.5	2.311.9	85.570.583.31	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 2 ▽ APRIL, 1930

DRINKING FOUNTAIN ON AN OREGON HIGHWAY

For sale by the Superintendent of Documents, Washington, D. C. - - - - - - - - - - - - - - - See page 2 of cover for price

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 11, No. 2 **APRIL, 1930** **R. E. ROYALL, Editor**

TABLE OF CONTENTS

Page

Drinking Fountains Along Oregon Highways 25

Flow of Flood Water Over Railway and Highway Embankments 30

Plans for International Road Congress Announced 35

Motor Vehicle Registrations and Revenue and Gasoline Taxes in 1929 36

Soil Sampling With a Compressed Air Unit 40

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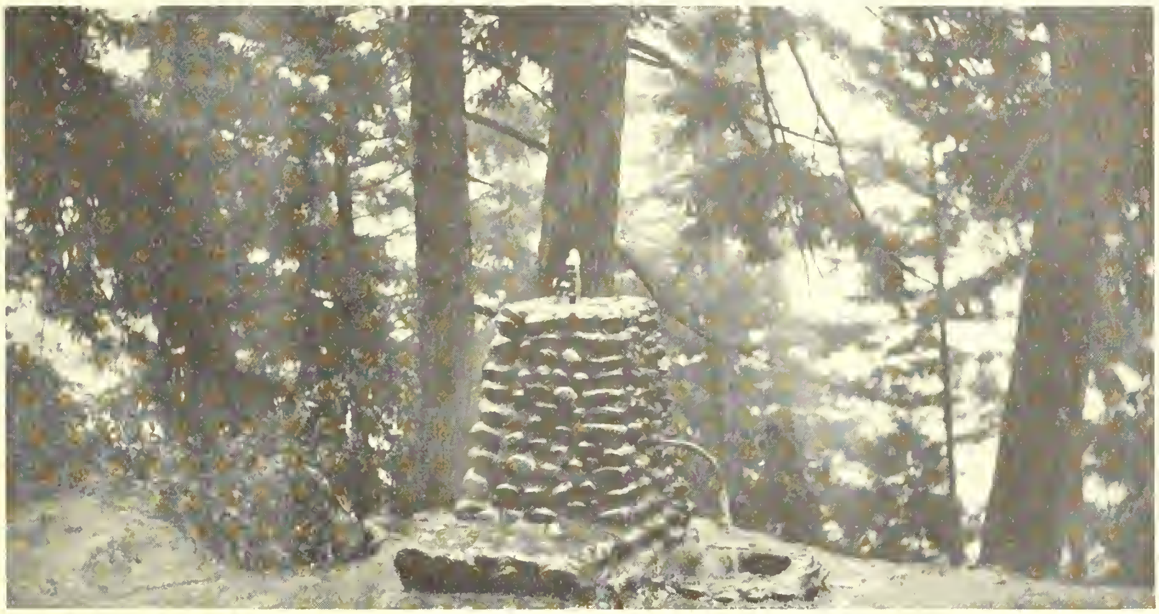
DRINKING FOUNTAINS ALONG OREGON HIGHWAYS

Reported by T. M. DAVIS, Highway Engineer, United States Bureau of Public Roads



ON THE ASHLAND-KLAMATH FALLS HIGHWAY

MOTORISTS driving along the highways in the State of Oregon will occasionally pass a sign notifying them that there is good drinking water 300 feet ahead. At such places they will find an artistically designed drinking fountain erected by the Oregon State Highway Commission and a water supply which they may feel safe in using.

No standard plans have been prepared for the construction of these fountains, each fountain being designed by the engineers to harmonize with the surroundings and to make use of the native materials at hand. Though the same general scheme is followed, the designs differ, being influenced by available materials, site, and quantity of water.

At the springs a small reservoir of concrete or cement rubble masonry is constructed, from which a pipe leads to the fountain. Each reservoir has overflow and drain pipes. The fountains are of native rock, some of them built of hand-faced stone, while others are of rubble masonry, using bowlders or large-size gravel.

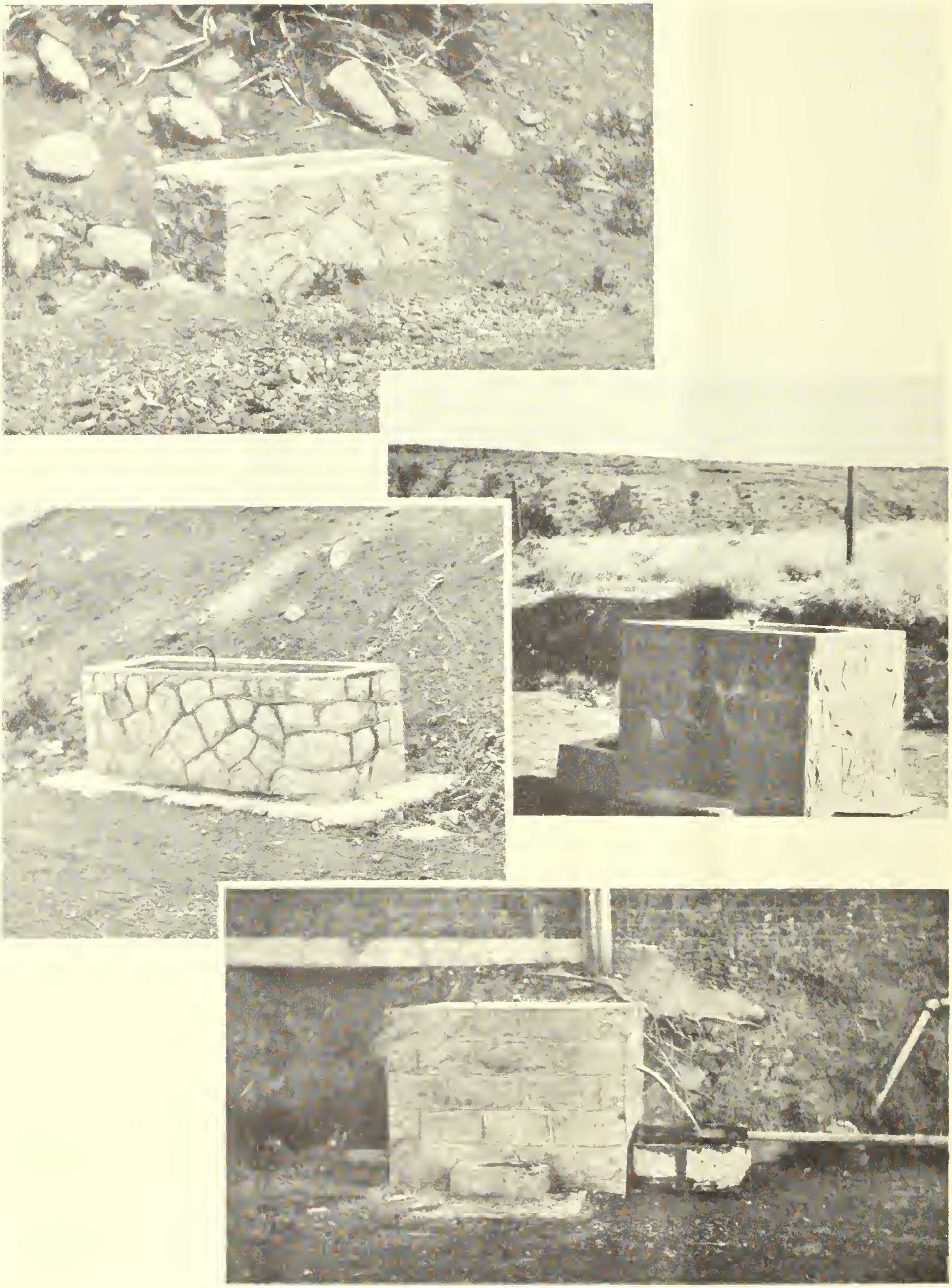
In most instances the fountains were constructed by the regular maintenance forces of the State highway department, though some were constructed by contractors' forces. In the erection of five fountains in Oregon by the United States Bureau of Roads, built according to the same general plans as the State fountains, it was found cement rubble masonry fountains cost from $200 to $300 each.

The fountains have bubblers for drinking, and a pool from which water for filling radiators may be dipped. When there is not a sufficient supply of water for bubblers a small pipe has been used for drinking purposes. Many of these fountains have been built with a step at one of the bubblers, making it more convenient for children. The pools are used mostly for obtaining water for filling radiators, and a few of the fountains have a small pool constructed near the ground for the use of dogs or other small animals. Shut-off and drain valves are used so that proper care of the fountain may be taken during severe freezing weather.

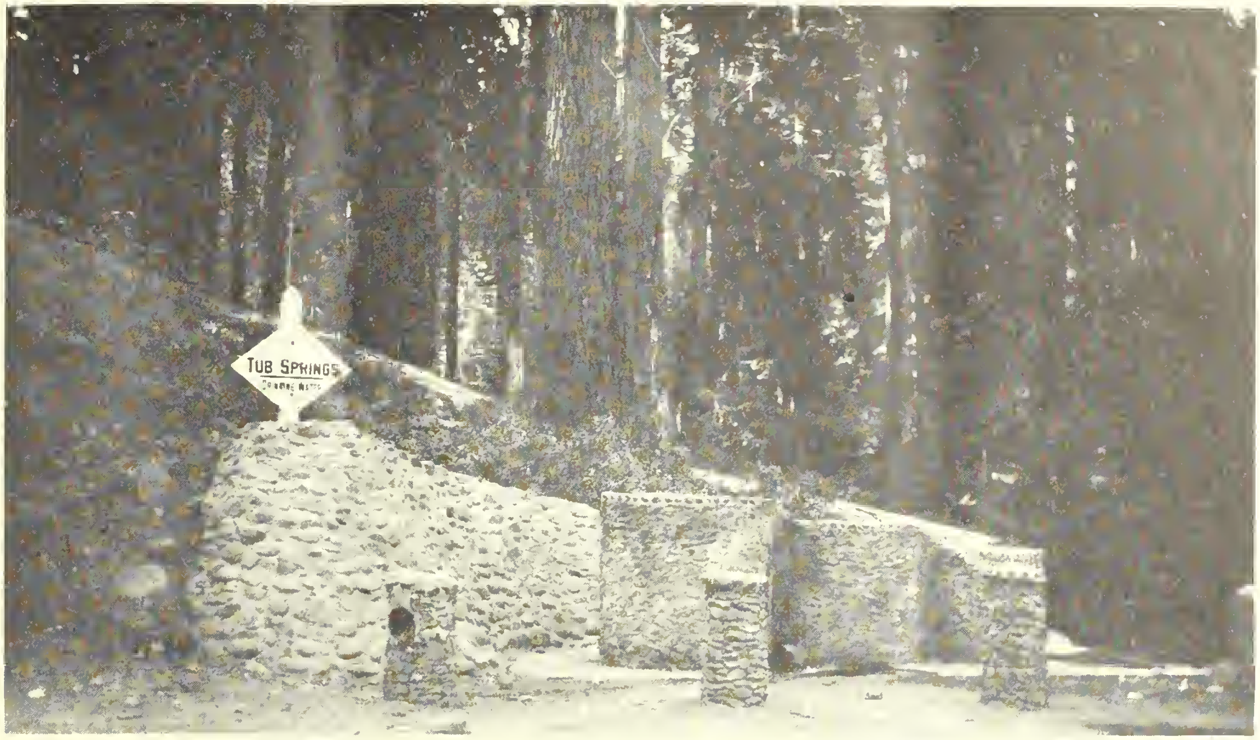
The water used is from live springs and is tested by the State to ascertain its suitability for drinking before the erection of a fountain. Signs are placed about 300 feet each side of the fountains, calling attention to drinking water ahead, and at the fountains themselves ample parking space is provided so that cars may be parked off the highway.

It is found that these fountains are a great convenience to motorists, and they are used by a great many people during the summer months, particularly on the heavier traveled highways. Many are situated at scenic locations with tall evergreen trees and beautiful shrubbery, ferns, and wild flowers; and motorists find it a joy to tarry for a few minutes' rest. During the summer days there will generally be found one or more cars at each fountain, the travelers resting a minute or two and obtaining a drink of cold spring water, or filling a hot radiator with fresh cool water.

At present there are some 30 fountains along the Oregon highways, and additional ones are placed wherever suitable conditions are found. The construction of these fountains by the Oregon State Highway Commission is in line with their policy of developing the scenic features of the highways of the State.



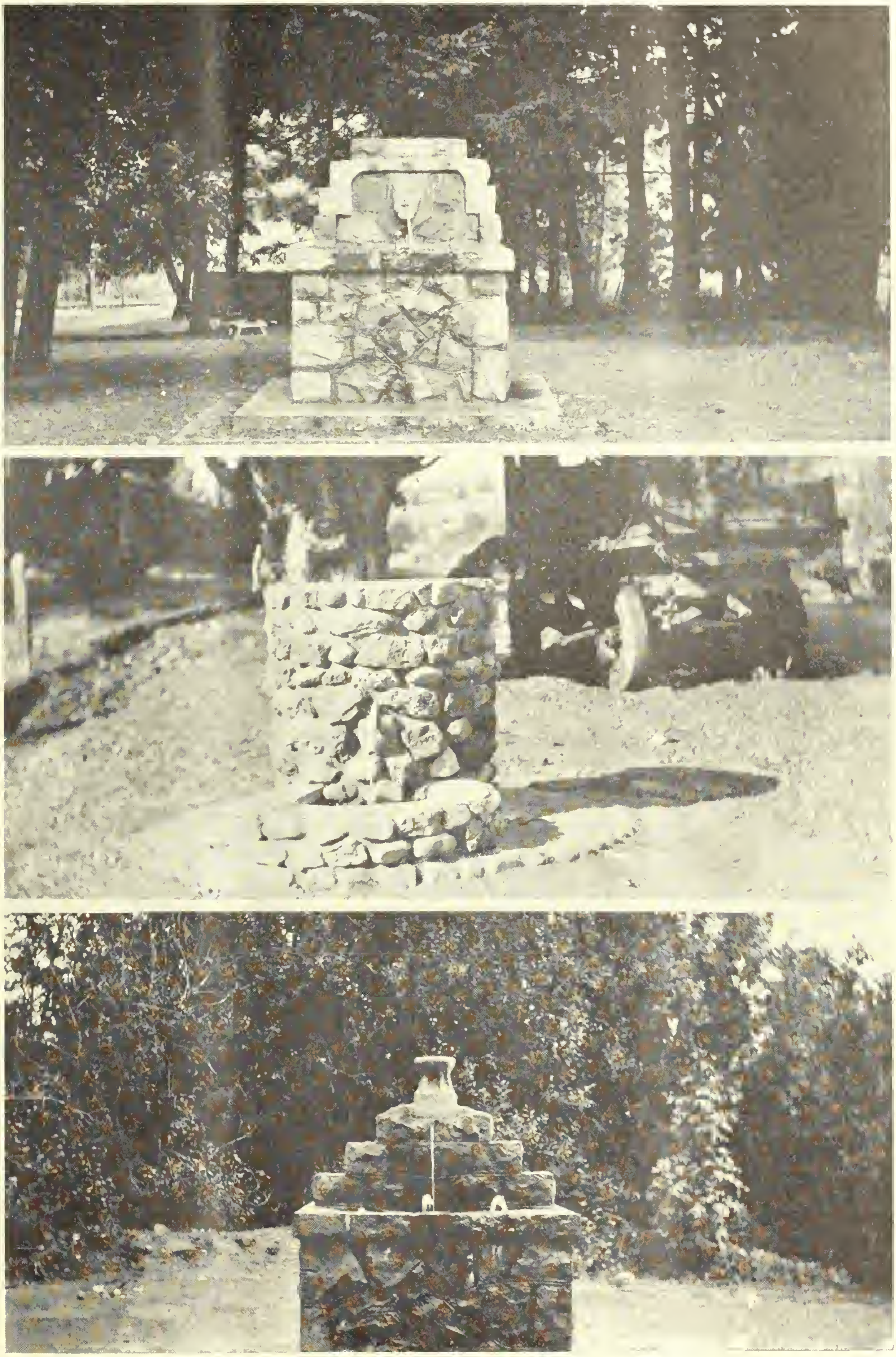
Drinking-fountain Designs are Varied According to Available Materials and the Water Supply



On Federal-aid Project No. 86 Between Ashland and Klamath Falls

On United States Highway Route No. 199 Near the California State Line

DRINKING FOUNTAINS ON OREGON HIGHWAYS



Many of the Fountains are in Attractive Locations Where Travelers Often Stop for a Short Rest

FLOW OF FLOOD WATER OVER RAILWAY AND HIGHWAY EMBANKMENTS

By DAVID L. YARNELL, Senior Drainage Engineer United States Bureau of Public Roads, and FLOYD A. NAGLER, Professor of Hydraulic Engineering, State University of Iowa

THIS report presents the results of 572 experiments on the flow of water over full-size sections of railway and highway embankments, made for the purpose of developing formulas for use in calculating the quantity of flood water flowing over such embankments. The experiments were conducted by the Bureau of Public Roads of United States Department of Agriculture and the State University of Iowa at the University hydraulic laboratory. The rails and ties used were furnished by the Cedar Rapids and Iowa City Railroad, J. D. Wardle, chief engineer.

THE PROBLEM OUTLINED

A knowledge of the quantity of water that flows down a given valley during extreme floods is often a matter of considerable importance to the engineer. It is one of the fundamental bases of design, not only of railroad and highway embankments across such valleys, but of works planned for the protection of agricultural land and of cities and towns. Engineers, therefore, should avail themselves of every opportunity to obtain reliable data on this suject.

Highway and railroad embankments across river valleys generally act as barriers to the flow of flood water and often are overtopped during the highest floods. At such an embankment the total discharge is the sum of that through bridge, trestle, and culvert openings, and that flowing over the embankment. The discharge through a bridge opening during a flood may be computed with a fair degree of accuracy if the drop-down at the bridge opening is known. A highway, railroad, or levee embankment over which water is flowing may be treated as a broad-crested weir. Therefore if the proper coefficient for a particular type of embankment is known the quantity of water flowing over an embankment of that type may be determined with reasonable accuracy. The specific purpose of this investigation was to determine these coefficients for certain types and widths of embankments so that the usual formulas for computing flow over broad-crested weirs could be employed.

THE MODEL EMBANKMENT DESCRIBED

The hydraulic laboratory of the University of Iowa is located on the west bank of the Iowa River, at the university dam. The principal testing canal is 190 feet long, 10 feet wide, and 10 feet deep. At the upstream end of the canal, where it joins with the end of the dam, is a wooden head gate 10 feet wide by 12 feet deep. A 10-foot weir of the suppressed type for measuring flow in the canal is located 60 feet downstream from the head gate. Numerous baffles were placed in the canal immediately below the head gate to obtain uniform velocity distribution as the water approached the weir, and a smooth flow over the crest. Similar baffles were placed in the canal immediately downstream from the weir to prevent commotion of the water as it approached the embankments.

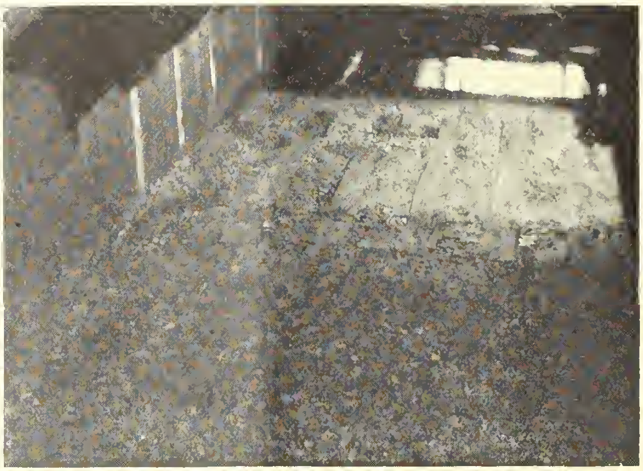


FIGURE 1.—SINGLE-TRACK EMBANKMENT WITHOUT RAILS

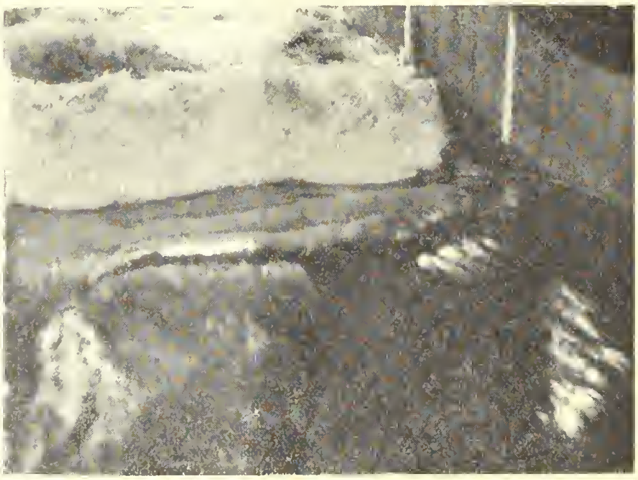


FIGURE 2.—SINGLE-TRACK EMBANKMENT WITHOUT RAILS, 50.4 CUBIC FEET PER SECOND FLOWING OVER EMBANKMENT, HEAD ON EMBANKMENT, 1.42 FEET

A full-size section of a single-track embankment, 10 feet in length, was built in the testing canal 92 feet downstream from the weir. The embankment proper consisted of a wooden structure which was covered with gravel and upon which was placed the standard depth of gravel ballast and the ties and rails. The American Railway Engineering Association specifications for standard single and double track, gravel ballast, were followed. Seventy-pound rails were used. The top of rail of the single track was 3.8 feet above the floor of the canal. A view of the single-track embankment without rails is shown in Figure 1. The same embankment under test is shown in Figure 2. The embankment without rails corresponds to a highway with approximately a 12-foot roadway.

In building the double track the extra track was added to the upstream side of the single track, the distance center to center of tracks being 13 feet. The height of the top of rail of the double track, however, was only 1.8 feet, as the floor here is 2 feet higher

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[1] Acknowledgment is made to Prof. S. M. Woodward, State University of Iowa, for valuable advice and suggestions, and to P. L. Hopkins, junior civil engineer, Bureau of Public Roads, and Nolan Page, graduate research assistant in hydraulics, State University of Iowa, who assisted in the laboratory work and in the computation of esults.

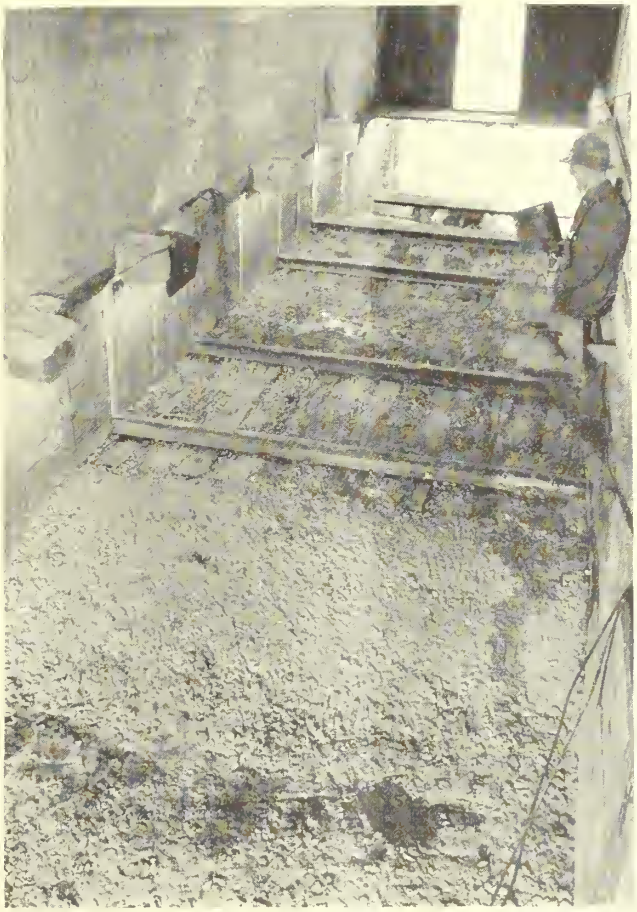

Figure 3.—Double-Track Embankment With Rails in Place

than the floor in the downstream end of the canal. This difference is due in part to a false wooden floor placed in the canal for use in connection with other experiments. The double-track embankment with rails in place is shown in Figure 3, and Figure 4 shows the double-track embankment while under test. Corresponding views with one track removed are shown in Figures 5 and 6. This embankment, without rails, is representative of a highway with a roadway 23 feet wide.

Twenty-seven staff gages, graduated to hundredths of a foot, were placed along the walls of the canal at frequent intervals, 14 being placed along the west wall and 13 along the east wall. The zero of these staffs was set level with the top of rail.

A bear-trap weir 6 feet high and located 22 feet downstream from the center of the single track, or 28.5 feet from the center of the double track, was used to regulate the water level downstream from the embankment, and in some experiments to submerge the embankment. This weir was hung on hinges and was regulated by means of a block and tackle attached to a windlass.

It will be noted in Figure 3 that the downstream face of the embankment is grouted. This was done to prevent the material from washing off of the wooden base.

The side slopes of most highway and railroad embankments are covered with a heavy growth of grass and weeds which resist erosion.

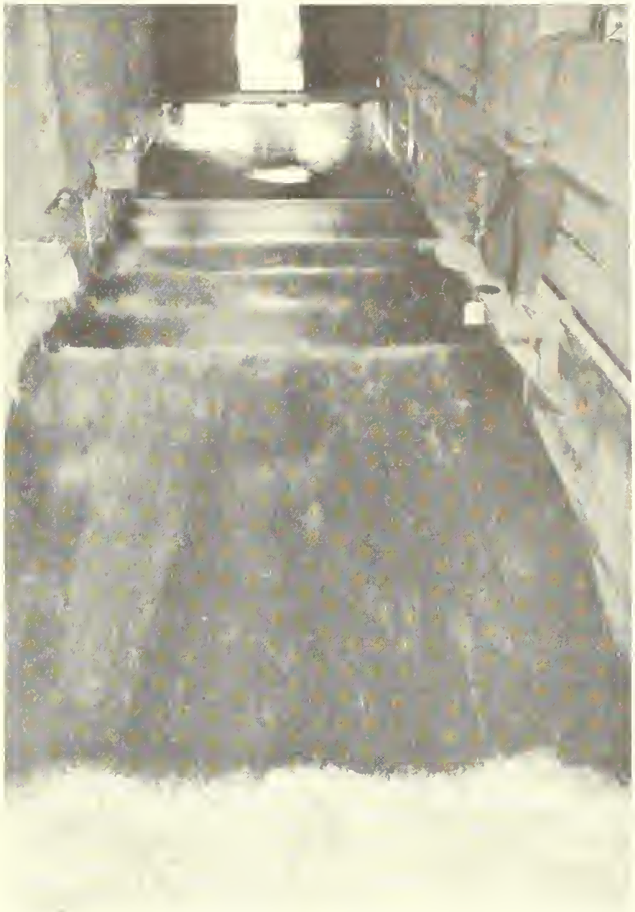

Figure 4.—Double-Track Embankment With Rails in Place, 46 Cubic Feet per Second Flowing Over Tracks. Head on Upstream Rail, 1.33 Feet. Standing Waves Caused by the Rails

THE TESTS

Tests on each embankment were begun with a head of 0.5 foot of water discharging over the measuring weir, and continued with successive increases of 0.1 foot in head on the weir, until the maximum flow was obtained. For each head on the measuring weir the following conditions of outflow were imposed by means of the bear-trap weir: (1) With water surface on the downstream side of the track raised until the ratio $\frac{d}{D}$ (depth of water over rail on the downstream side of the embankment, to depth on upstream side of embankment as illustrated on drawings) was about 0.95; (2) with $\frac{d}{D}$ less than 0.95 (obtained by gradually lowering the bear-trap weir); (3) with water discharging freely over the embankment, thus simulating free flow over one type of broad-crested weir.

In each test a constant head was first obtained on the measuring weir and then the bear trap was adjusted to obtain a definite ratio of depth of flow. When this had become constant, one observer first took readings on the weir hook gage and weir staff, then successively read the staffs downstream on one side of the channel and upstream on the other side. After determining that the readings on the two opposite upstream staffs

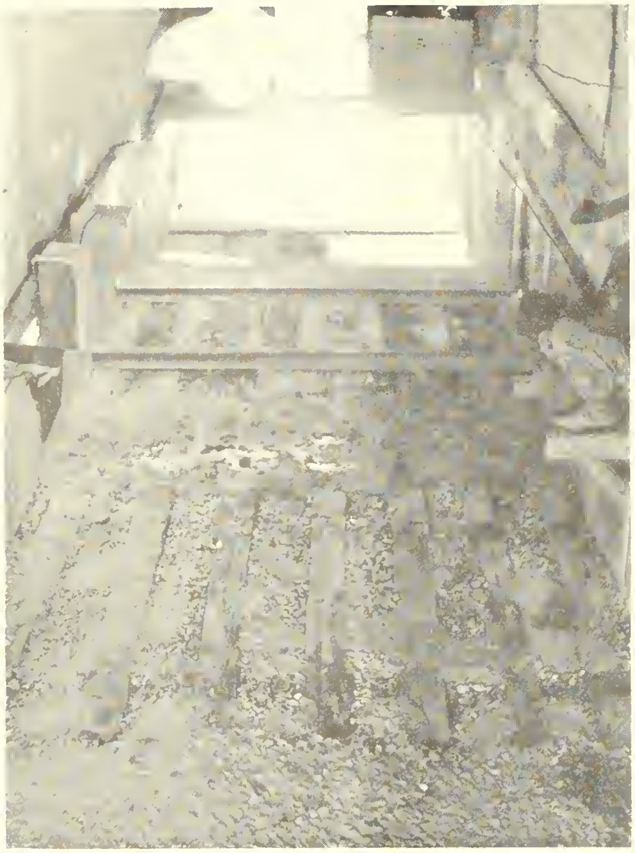

Figure 5.—Double-Track Embankment With Upstream Track in Place and Downstream Track Removed

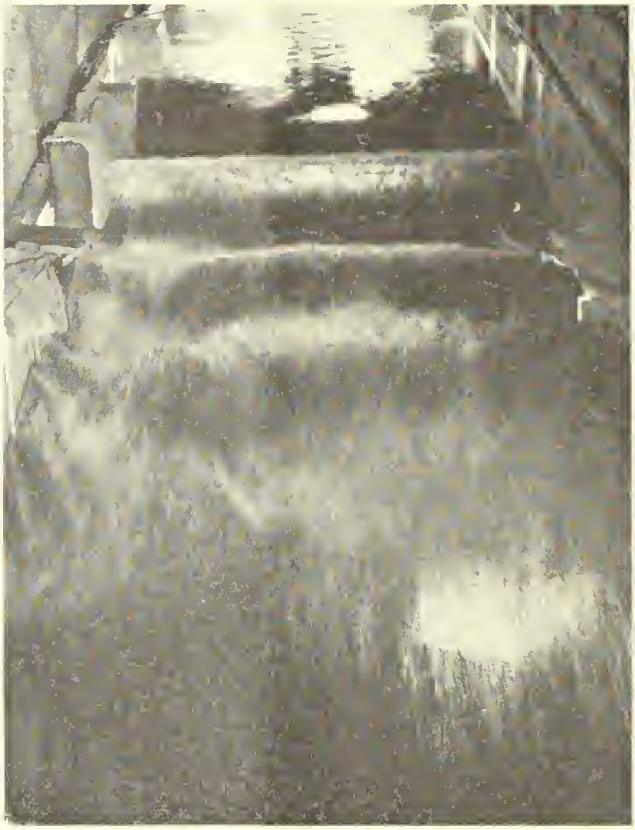

Figure 6.—Double-Track Embankment With Upstream Track in Place and Downstream Track Removed. 42.5 Cubic Feet per Second Flowing Over Track. Depth of Flow, 1.12 Feet Over Track

checked, he again took readings on the weir hook gage and weir staff. In the meantime, another observer plotted a profile of the water surface. Horizontal lines 0.1 foot apart, as well as vertical lines 1 foot apart, had been painted on both sides of the canal. By means of these continuous gage lines (shown in fig. 3) an accurate profile of the many variations in the water surface was obtained which it was not possible to obtain with the staff gages. This method gave a record of the waves caused by the rails as illustrated in Figures 4 and 6.

In testing the single-track embankment with rails in place it was noted that a standing wave was sometimes formed over the downstream rail. To determine the effect of the downstream rail on the flow some tests were run with only the upstream rail in place. On the double-track embankment tests were also run with the upstream track in place and the downstream track removed.

The results of the tests were computed as the experiments were under way, thus making it possible to rerun any doubtful tests.

FORMULAS DERIVED FOR VARIOUS CONDITIONS OF FLOW

The following basic formula was used in all of the computations:

$$Q = CLH^{3/2} \qquad (1)$$

In this formula,

Q = volume of discharge per unit of time,
C = an empirical coefficient,
L = length of the embankment overflowed,
H = the head corrected for the effect of the velocity of approach; i. e., measured depth plus $\frac{V}{2}$ where V is the mean velocity of approach to the weir.

The discharge over the measuring weir was compute by Bazin's weir formula. The readings on the various staffs gave the heights of the water above the top of rail. In the tests on the embankments with the rails removed the height of the rail was therefore added to the staff readings to get the depth of water on the embankment. For the tests in which the embankment was submerged the degree of submergence is expressed by the ratio $\frac{d}{D}$ before mentioned.

The results of the experiments were plotted on cross-section paper with the ratio $\frac{d}{D}$ as abscissas and the empirical coefficient C as ordinates. These curves are shown in Figures 7 to 12, inclusive. It will be noted that when $\frac{d}{D} = 0$, the water flowing over the embankment has a free drop. Under such conditions the coefficient C varies more for some embankments than for others. For example, in Figure 11 the points are grouped whereas in Figure 8 they are more scattered. Since it was impossible to show all such values of the coefficient on the diagrams (many of them coincided), the points through which the curves are drawn were

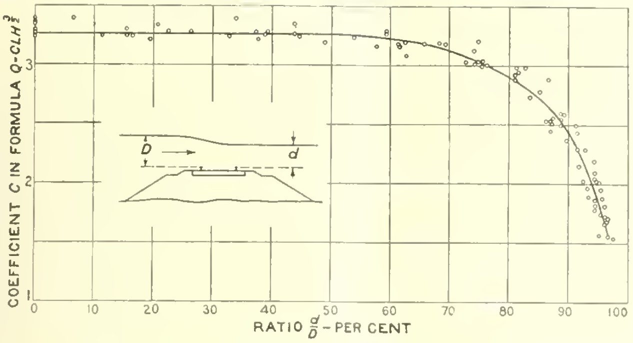

Figure 7.—Relation Between Coefficient and Per Cent Submergence, Single Track, Both Rails in Place

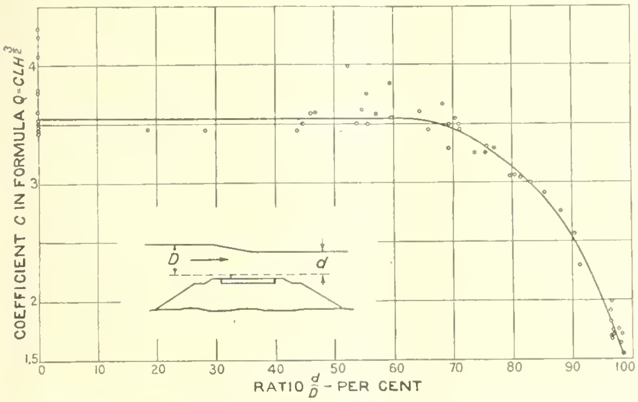

Figure 8.—Relation Between Coefficent and Per Cent Submergence, Single Track, Downstream Rail Removed

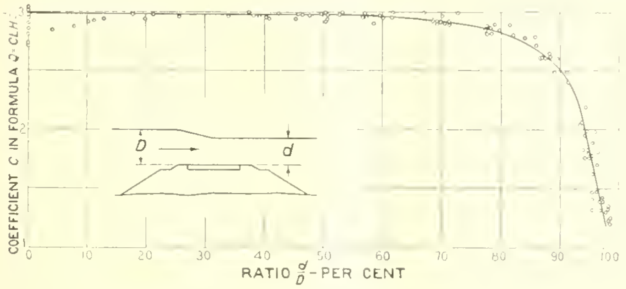

Figure 9.—Relation Between Coefficient and Per Cent Submergence, Single Track, Both Rails Removed. Equivalent to a Highway Embankment With a 12-Foot Roadway

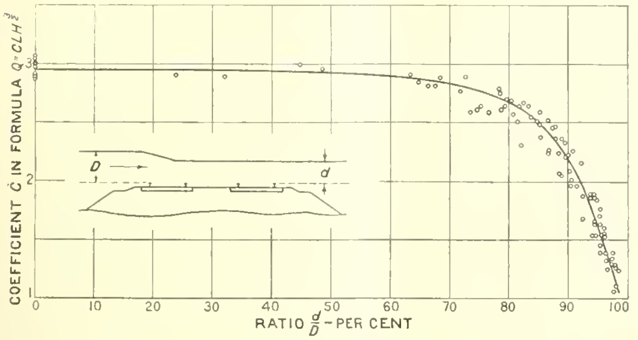

Figure 10.—Relation Between Coefficient and Per Cent Submergence, Double Track, Four Rails in Place

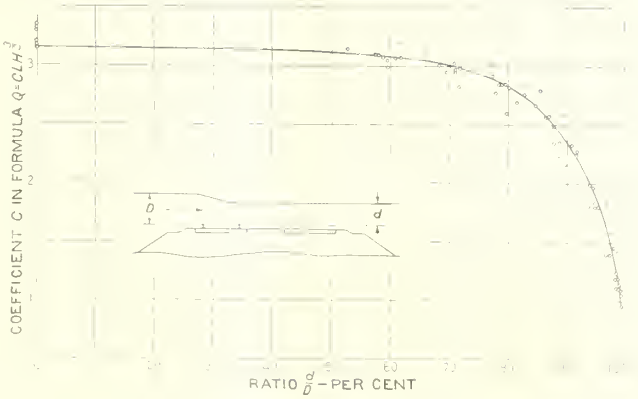

Figure 11.—Relation Between Coefficient and Per Cent Submergence, Double Track, Two Downstream Rails Removed

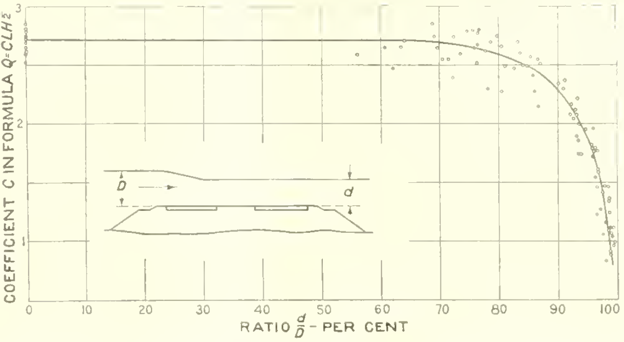

Figure 12.—Relation Between Coefficient and Per Cent Submergence, Double Track, All Four Rails Removed. Equivalent to a Highway Embankment With a 23-Foot Roadway

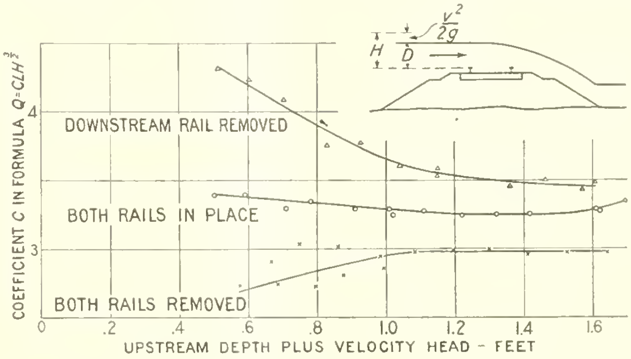

Figure 13—Variation of Coefficient With Head. Single Track

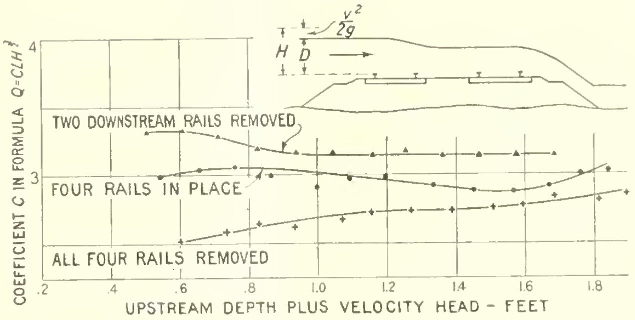

Figure 14.—Variation of Coefficient With Head. Double Track

determined in the following manner: Values, the discharges, Q, were plotted on logarithmic paper as ordinates and the heads, corrected for velocity of approach, as abscissas. A line having a slope of 3/2 was drawn through the majority of the points, the greater weight being placed on the higher heads. The intercept on the unity vertical axis was taken as the value of C through which the curve should pass.

During the progress of the tests it was observed that for values of $\frac{d}{D}$ less than 0.6, the coefficient was practically constant since the elevation of the tailwater had no effect on the height of the headwater. This was due to the fact that the critical velocity occurred at the downstream edge of the embankment. Therefore, to conserve time in experimenting on some of the embankments, comparatively few tests were run with values of $\frac{d}{D}$ between 0.1 and 0.5.

For those tests in which the water had a free drop over the embankment it was noted that the coefficient varied somewhat with the head when the exponent of H was taken as 3/2. This variation is shown in Figures 13 and 14 in which the coefficients have been plotted as ordinates and the heads as abscissas. These curves show that the exponent of H is not exactly 1.5. The test data were therefore plotted on logarithmic paper with the discharge Q as ordinates and the head H (corrected for velocity of approach) as abscissas. Lines were drawn through the points and the discharge equations determined for the various types of embankments.

The equations as determined for free flow over the embankments are as follows:

Single track, both rails in place—

$$Q=3.27\ LH^{1.47} \tag{2}$$

Single track, upstream rail only in place—

$$Q=3.66\ LH^{1.37} \tag{3}$$

Single track, both rails removed—

$$Q=3.00\ LH^{1.49} \tag{4}$$

Double track, all four rails in place—

$$Q=2.95\ LH^{1.47} \tag{5}$$

Double track, upstream track only in place—

$$Q=3.17\ LH^{1.48} \tag{6}$$

Double track, all four rails removed—

$$Q=2.66\ LH^{1.60} \tag{7}$$

USE OF FORMULAS ILLUSTRATED BY EXAMPLES

These experiments have made available coefficients for use in hydraulic formulas for computing the flow over embankments during floods. When free flow exists over an embankment one of formulas (2) to (7) may be used, depending upon the type of the embankment. If the embankment is submerged the appropriate coefficient may be taken from the curves shown in Figures 7 to 12, after the percentage of submergence has been determined. In all of the computations for the curves and formulas the head corresponding to the velocity of approach has been added to the depth of water over the embankment to get the true head. Thus these coefficients are applicable to all embankments regardless of the velocity of approach. In a specific case, however, the velocity of approach must be assumed in the preliminary calculations and the assumed value checked after the discharge has been computed. This procedure can best be illustrated by practical examples.

Example 1.—Assume that water has been discharging over a single-track railroad embankment. The track at elevation 75 was not washed out. After the flood a survey for high-water marks showed that the water on the upstream side of the track had been at elevation 76.6, and on the downstream side of the track at elevation 74. Assume the height of the embankment (to top of rail) above the natural ground surface to be 6.5 feet, and for convenience consider a length of 100 feet of track. The formula applicable to this problem is equation (2) which is

$$Q=3.27\ LH^{1.47}$$

in which

$$L=100 \text{ feet.}$$

The measured head is 1.6 feet. Assume the velocity of approach to be 0.80 foot per second, then

Velocity head = 0.0099
Total head, $H=1.61$

and

$$Q=3.27\times100\times1.61^{1.47}$$

or

$$Q=658 \text{ cubic feet per second.}$$

But the cross-sectional area of flow is 810 square feet and

$$\frac{Q}{A}=\frac{658}{810}=0.81 \text{ foot per second,}$$

which approximates the assumed velocity of approach.

Example 2.—A single-track railroad was overtopped by flood water. A survey showed the track to be at elevation 50, the water level upstream at 53, and downstream at 52.5. The height of the embankment or top of rail is 3.5 feet above the natural ground surface, and the length of track overflowed 100 feet. The percentage of submergence would then be

$$\frac{d}{D}=\frac{2.5}{3.0}=83.3.$$

From Figure 7 the coefficient, C, is found to be 2.80. Hence the formula would be

$$Q=2.80\ LH^{3/2}$$

in which

$$L=100.$$

The measured head is 3 feet. Assuming the velocity of approach to be 2 feet per second, the velocity head is 0.0622 and the total head, H, is 3.0622.

Thus

$$Q=2.80\times100\times3.0622^{3/2}$$
$$Q=1{,}500.$$

The area of flow is 650 square feet and the velocity of approach is $\frac{1{,}500}{650}=2.31$ feet per second. The first assumption being too small, assume the velocity of approach as 2.3 feet per second. The velocity head then is 0.0822; and the total head, $H=3.0822$.

$$Q=2.80\times100\times3.0822^{3/2}$$
$$Q=1{,}515.$$

The velocity of approach is $\frac{1{,}515}{650}=2.33$ feet per second which agrees quite closely with the second assumption. Thus the discharge over a 100-foot section of the track is 1,515 cubic feet per second, or 15.15 cubic feet per second per lineal foot of track overflowed.

PLANS FOR INTERNATIONAL ROAD CONGRESS ANNOUNCED

The American Organizing Commission announces that the Sixth International Road Congress will be held in this country in October, 1930, at the invitation of the United States Government, with the object of continuing the studies begun in Paris in 1908 and continued in Brussels in 1910, in London in 1913, in Seville in 1923, and in Milan in 1926.

The congress will open in Washington on Monday, October 6, and close on Saturday, October 11.

Simultaneously with the congress it is contemplated that a private international road exhibition will be held by the American Road Builders' Association.

TENTATIVE PROGRAM OF THE CONGRESS

The official opening of the congress will occur Monday afternoon, October 6, at Constitution Hall, preceded by a meeting in the morning of the International Permanent Commission at the offices of the Chamber of Commerce of the United States.

Registration will begin Monday morning and continue through Thursday, October 9. Members are urged to register as early during the week as possible.

All sessions of the congress, except the official opening, will be at the building of the Chamber of Commerce of the United States.

Section meetings will be held the morning and afternoon of October 7, the morning sessions being followed by motion-picture exhibits of road-building operations.

The same procedure will hold for Wednesday, October 8, and for Thursday, October 9.

Friday is reserved for a visit to the experiment station of the United States Bureau of Public Roads, and over Mount Vernon Memorial Highway to Mount Vernon, home of George Washington.

Saturday morning, October 11, will see the official closing of the congress, with the afternoon of that day reserved for excursions, to be followed by sight-seeing tours on Sunday, October 12.

The detailed program will be announced at a later date.

AGENDA

(Prepared by the Permanent International Commission)

First Section

CONSTRUCTION AND MAINTENANCE

First question.—Results obtained by the use of:
(*a*) Cement;
(*b*) Bricks or other artificial paving.

(Methods employed for road construction and maintenance in these materials.)

Second question.—The most recent methods adopted for the use of tar, bitumen, and asphalt in road construction.

Third question.—The construction of roads in new countries, such as colonies and undeveloped regions.

Second Section

TRAFFIC AND ADMINISTRATION

Fourth question.—Ways and means of financing highways:
(*a*) Road construction;
(*b*) Maintenance.

Fifth question.—Highway transport: Correlation and coordination with other methods of transport; adaptation to collective (organizations) and individual uses.

Sixth question.—(1) Traffic regulation in large cities and their suburbs; traffic signals; design and layout of roads and adaptation to traffic requirements in built-up areas. (2) Parking and garaging of vehicles.

There probably will be a number of unofficial luncheons and dinners tendered the delegates and members by various organizations. The jurisdiction of the American Organizing Commission, however, is limited to the official events appearing on the program.

CONDITIONS FOR PARTICIPATION IN THE CONGRESS

In addition to the delegates of the various countries, expressly appointed by their respective Governments, the permanent and temporary members of the Permanent International Association of Road Congresses may take part in the Congress.

Permanent members.—The permanent members of the association are members who have the right to participate in all the road congresses. Individuals or representatives of groups already enrolled as permanent members are required only to notify the American Organizing Commission at Washington of their intention to participate in the sixth congress. (Blank forms for this purpose will be supplied upon request.)

Individuals or groups desiring to be enrolled as permanent members of the association should request suitable application forms from one of the addresses given below.[1]

For individual permanent members the regular annual dues are 25 French francs ($1). These dues, however, are increased to 125 French francs ($5) for new members enrolled during a congress year, as in the case for the year 1930. For life membership the subscription fee is 500 French francs ($20).

For groups, the minimum annual dues in any year are 100 French francs ($4). For each 100 French francs paid, one delegate may be sent to the congress. All fees received from the above classes of membership are paid to the Permanent International Association of Road Congresses.

Temporary members.—Individuals or groups who do not belong to the association and who desire only to participate in the sixth congress, are considered as temporary members of the association, under articles 2, 8, and 10 of the statutes. Honorary members are enrolled as temporary members.

The fee for enrollment in the congress for each temporary member is 125 French francs ($5). The payment of 250 French francs ($10) bestows the right to the title of honorary member. Three-fifths of the fees received from temporary members are allocated to the American Organizing Commission and two-fifths to the Permanent International Association of Road Congresses.

Applications for temporary membership should be made on a form which can be obtained from any of the addresses given in footnote 1.

Membership privileges.—All classes of members mentioned above have the right to attend all the meetings of the congress, and to participate in all official receptions and official excursions during the congress proper.

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[1] The address of the secretary general of the Permanent International Association of Road Congresses is M. Le Gavrian, secretary general, Permanent International Association of Road Congresses, 1, Avenue d'Iéna, Paris, XVI, France. The address of the secretary general of the American Organizing Commission is Thos. H. MacDonald, secretary general, American Organizing Commission, 1723 N Street NW, Washington, D. C., U. S. A. The address of the secretary of the British Organizing Committee is E. B. Hart, secretary, British Organizing Committee, 7 Whitehall Gardens, London, S. W. 1, England

They are also entitled to the special transportation rates established for attendance at the congress.

They will receive, previous to the opening of the congress, the official papers on the subjects indicated in the program, and after the congress a copy of the proceedings. These publications can be obtained only through membership in the congress.

The executive bureau, and the American Organizing Commission, however, do not undertake to send the papers previous to the opening of the congress to those members who have not been enrolled at least one month prior to the opening. (Art. 11 of the statutes of the association.)

A circular is available for distribution giving information concerning reductions in rates of ocean steamship lines, passports, and hotel accommodations and rates. Hotel reservations may be made through the American Organizing Commission.

A railroad rate of three-fourths the regular round-trip fare has been arranged for members attending the congress.

MOTOR VEHICLE REGISTRATIONS AND REVENUE AND GASOLINE TAXES IN 1929

According to reports received from State registration authorities by the Bureau of Public Roads, a total of 26,501,443 motor vehicles were registered in 1929. From the owners of these vehicles the States and the District of Columbia collected in license fees, registration fees, permit fees, fines, etc., the sum of $347,843,543.

The registration figure includes passenger automobiles, taxis, buses, motor trucks and road tractors, trailers, and motor cycles, and represents an increase of 2,008,319 or 8 per cent over the 1928 figure. The total fees collected represent an increase of $25,213,518 over the 1928 figure. After deducting $24,505,737 for collection and miscellaneous purposes, the balance of $323,337,806 was applied to highway purposes; $223,292,969 to State funds, $66,861,364 to local funds, and $33,183,473 to State and county bond funds.

The States having the 10 highest registration figures are as follows: New York, 2,263,259; California 1,974,341; Ohio, 1,766, 614; Pennsylvania, 1,733,283; Illinois, 1,615,088; Michigan 1,395,102; Texas, 1,348,107; Indiana, 866,715; New Jersey, 832,332; and Massachusetts, 817,704.

In percentage gain, the District of Columbia and New Mexico each show 19 per cent. Nevada reports a gain of 16 per cent, Arizona 15 per cent, and Utah 14 per cent. Four States, Georgia, Maryland, Massachusetts, and Tennessee, each show a gain of 12 per cent, and three States, Michigan, Montana, and Texas each report an increase of 11 per cent. California, Idaho, Kentucky, New Jersey, and Washington each show a gain of 9 per cent.

Details as to the number of registrations are shown on page 37 and registration receipts are shown on page 38.

GASOLINE TAX YIELDS LARGE REVENUE FOR HIGHWAYS

Reports received by the Bureau of Public Roads show that $431,636,454 were collected in taxes on the sale of 13,400,180,062 gallons of motor fuel in 1929. These figures include a 12-month collection in 46 States and the District of Columbia, a 5-month collection in Illinois, and the collections of eight months in New York. The entrance of the latter two States into the list of those collecting a gasoline tax marks the adoption of this method for part payment of the highway bill by all States. The pioneer States—Oregon and Colorado—led the way in 1919. All others have followed in the succeeding period of 11 years, but the tax did not become effective in New York until May 1 and in Illinois until August 1.

The average fee per gallon was 3.22 cents as against 3 cents in 1928, 20 States having increased the rate of taxation either 1 or 2 cents. The highest tax per gallon was 6 cents; the lowest 2 cents. At the close of the year, 3 States had a 6-cent tax; 8 a 5-cent tax; 19 a 4-cent tax; 1, Utah, a 3½-cent tax; 10 a 3-cent tax; and 7 States and the District of Columbia a 2-cent tax.

During the year the rate per gallon was increased 1 cent in Colorado, Florida, Indiana, Kansas, Minnesota, North Carolina, North Dakota, Ohio, Oklahoma, Pennsylvania, South Carolina, Vermont, Washington, and Wyoming; a 2-cent increase became effective in Georgia, Louisiana, Montana, Nebraska, Tennessee, and Texas.

Comparison of the total number of vehicles registered in 1929, with the total tax collected and with the taxable gallonage in all States (except New York and Illinois) and in the District of Columbia shows an average tax revenue of $17.72 per vehicle and an average purchase of 532 gallons of gasoline.

After deducting collection costs, the entire net revenue in 34 States was used for construction and maintenance of rural roads. In the other 14 States and the District of Columbia, a total of $24,405,207 was used for other purposes. In three States a portion was used for public-school purposes. In eight States, a part of the revenue went to cities for repair and improvement of streets, as did the entire collection for the District of Columbia. In six States, small sums were deposited in general funds; in Mississippi, a special extra tax was collected in two counties for seawall protection of highways; and in New Jersey a small portion of the receipts was turned over to the department of commerce and navigation.

Of the portion of the total revenue applied to rural road purposes, $297,967,756 was used for construction and maintenance of State highways; $85,113,708 for construction and maintenance of local roads; and the balance of $23,371,785 was used for payments on State and county road bonds.

The table on page 39 shows the total tax earnings and total number of gallons taxed in the various States.

MOTOR VEHICLE REGISTRATIONS, 1929 [1]

[Compiled for calendar year from reports of State authorities]

State	Registered motor vehicles individually and commercially owned [2]			Other registered vehicles		Tax-exempt official motor cars and motor cycles			Number of licenses or permits		Total registered motor cars and trucks, in 1928	Year's change in motor-vehicle registrations		State
	Grand total registered motor cars and trucks	Passenger automobiles, taxis, and buses [3]	Motor trucks and road tractors	Trailers [4]	Motor cycles	United States cars [5]	State and local cars	Motor cycles (official)	Dealers	Operators and chauffeurs		Number increase or decrease (−)	Per cent	
Alabama	285,533	247,701	37,832	3,024	704	167	869		4,712	2,714	269,519	16,014	6.0	Alabama.
Arizona	109,013	98,327	10,686	1,249	396	176	1,173	43	182	20,643	94,372	14,641	15.5	Arizona.
Arkansas	233,128	193,396	39,732	2,808	400	39	919	31	515	5,023	214,931	18,197	8.5	Arkansas.
California	1,974,341	1,760,308	214,033	42,899	9,628	1,217	15,005	552	14,000	250,855	1,799,890	174,451	9.7	California.
Colorado	303,489	274,988	28,501	159	1,142	283			3,684	7,916	284,867	18,622	6.5	Colorado.
Connecticut	328,063	278,057	50,006	324	2,314	71	2,068	270	4,073	379,122	309,792	18,271	5.9	Connecticut.
Delaware	54,960	44,728	10,232	389	308	44			684	60,887	51,210	3,750	7.3	Delaware.
Florida	345,977	288,684	57,293	900	1,104	75	3,788	204	1,906	2,326	352,961	−6,984	−2.0	Florida.
Georgia	358,905	310,362	48,543	2,597	1,138	934				2,973	318,856	40,049	12.6	Georgia.
Idaho	118,074	104,398	13,676	485	355	103	1,092	15	466	780	108,154	9,920	9.2	Idaho.
Illinois	1,615,088	1,411,753	203,335	5,068	6,055	979			4,605	106,551	1,504,359	110,729	7.4	Illinois.
Indiana	866,715	741,366	125,349	10,407	2,983	3,184	4,965		2,794	55,164	823,806	42,909	5.2	Indiana.
Iowa	784,450	714,919	69,531	528	1,650	44	3,461	76	2,483	19,067	733,466	50,984	6.9	Iowa.
Kansas	581,223	507,529	[6] 73,694	360	1,178	192	2,470	60	2,784		533,799	47,424	8.9	Kansas.
Kentucky	332,848	298,716	34,132		727	90	1,916		1,158	11,859	304,231	28,617	9.4	Kentucky.
Louisiana	280,868	234,565	46,303	8,000	600	209			456	22,834	264,293	16,575	6.3	Louisiana.
Maine	184,506	147,962	36,544	1,652	1,258	64	1,467	82	1,151	226,087	172,638	11,868	6.9	Maine.
Maryland	319,873	281,034	[7] 38,839	876	1,986	1,969			7,231	82,714	285,311	34,562	12.1	Maryland.
Massachusetts	817,704	719,436	98,268	556	5,568	556	900		3,094	944,338	726,295	91,409	12.6	Massachusetts.
Michigan	1,395,102	1,219,158	[6] 175,944	31,577	3,985	371			2,282		1,249,221	145,881	11.7	Michigan.
Minnesota	730,399	630,703	99,696	4,783	1,900	252	1,304		2,160		673,573	56,826	8.4	Minnesota.
Mississippi	250,011	217,362	32,649	4,321	204	74			5,567		246,242	3,769	1.5	Mississippi.
Missouri	756,680	671,237	[6] 85,443	2,686	1,893	311	1,604	5	2,686	33,265	712,965	43,715	6.1	Missouri
Montana	140,387	115,285	25,102		234	229			537	414	126,035	14,352	11.4	Montana.
Nebraska	418,226	375,946	42,280	6,515	972	226	1,101		3,584		391,355	26,871	6.9	Nebraska.
Nevada	31,915	25,302	6,613	273	95	42	468		119		27,376	4,539	16.6	Nevada.
New Hampshire	108,880	94,900	13,980	635	1,263	22			561	129,539	102,644	6,236	6.1	New Hampshire.
New Jersey	832,332	698,959	133,373	2,424	6,531	708	7,173	933	3,418	971,235	758,430	73,902	9.7	New Jersey.
New Mexico	78,374	76,000	2,374	509	203	156	820	4	208		65,737	12,637	19.2	New Mexico.
New York	2,263,259	1,922,068	341,191	8,232	13,527	1,666	17,499	1,313	5,039	2,708,036	2,083,942	179,317	8.6	New York.
North Carolina	483,602	430,651	52,951	3,584	1,298	429	7,092		7,614		464,376	19,226	4.1	North Carolina.
North Dakota	188,046	162,092	25,954		230	3			869		173,525	14,521	8.4	North Dakota.
Ohio	1,766,614	1,560,182	206,432	15,000	9,180	2,362	10,843		29,298	4,436	1,649,699	116,915	7.1	Ohio.
Oklahoma	570,791	510,401	60,390		1,196	530	4,725		3,923		529,843	40,948	7.7	Oklahoma.
Oregon	269,007	247,131	21,876	1,308	1,694	141	1,483	102	603	52,451	248,118	20,889	8.4	Oregon.
Pennsylvania	1,733,283	1,515,875	217,408	5,067	13,375	1,383	1,057		29,599	2,153,944	1,642,207	91,076	5.5	Pennsylvania.
Rhode Island	134,009	114,010	19,999	87	917	56	837	107	330	157,474	125,698	8,311	6.6	Rhode Island.
South Carolina	231,274	205,683	25,591	2,048	451	91	3,017		628		216,805	14,469	6.7	South Carolina.
South Dakota	204,199	181,419	[6] 22,780		207	85	999		1,110		191,374	12,825	6.7	South Dakota.
Tennessee	362,431	329,697	[6] 32,734		1,203	132	4,377		711		322,137	40,294	12.5	Tennessee.
Texas	1,348,107	1,165,150	182,957	16,260	4,016	2,505			4,423	14,897	1,214,297	133,810	11.0	Texas.
Utah	112,661	95,661	17,000		525	173	1,200				98,541	14,120	14.3	Utah.
Vermont	93,030	84,471	8,559	280	487	28			365	95,387	86,231	6,799	7.9	Vermont.
Virginia	387,205	328,525	58,680	966	2,141	1,141	3,514	143	4,561	10,376	360,545	26,660	7.4	Virginia.
Washington	442,341	379,995	62,346	2,986	2,403	637	4,349	180	5,109	460,622	402,875	39,466	9.8	Washington.
West Virginia	268,888	228,715	40,173	725	1,397	33	2,388	120	11,854	85,514	251,556	17,332	6.9	West Virginia.
Wisconsin	793,502	689,447	104,055	497	2,723	92	943	128	3,138		742,135	51,367	6.9	Wisconsin.
Wyoming	60,680	51,880	8,800		92	209	356		338		56,336	4,344	7.7	Wyoming.
District of Columbia	151,450	135,455	15,995		1,009	837	1,586	183	2,107	63,921	126,556	24,894	19.7	District of Columbia.
Total	26,501,443	23,121,589	3,379,854	193,044	114,845	[8] 33,179	118,828	4,551	188,719	9,143,364	24,493,124	2,008,319	8.2	Total.

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[1] This table lists only the number of registrations, licenses, and permits. For financial statement see table on next page.

[2] The first 3 columns show regularly registered motor cars and trucks, eliminating reregistrations, nonresident registrations, etc.

[3] Buses are included with passenger cars except as noted in next column.

[4] Some States do not register trailers.

[5] Bureau of the Budget data as of 1925.

[6] Buses included with trucks and not reported separately.

[7] Includes commercial cars with pneumatic tires, formerly reported with passenger cars.

[8] Total includes 7,859 "cars-at-large," not allocated to any State.

MOTOR VEHICLE REGISTRATION FEES, LICENSES, PERMITS, FINES, ETC., 1929[1]

[Compiled from reports of State authorities]

State [2]	Total gross receipts	Registration receipts [2]					Miscellaneous receipts [2]			Disposition of gross receipts					State [2]
		Motor car receipts			Other vehicles					Collection and administration [3]	For highway purposes			For other purposes	
		Total motor cars	Passenger cars and buses	Trucks and tractors	Trailers	Motorcycles	Dealers' licenses	Chauffeur and operator permits	Other miscellaneous		State highways	Local roads	State and county road bonds [4]		
Alabama [2]	$3,736,380									$177,819	$1,324,322	$728,594	$1,505,645		Alabama.[2]
Arizona	748,565	$526,390	$300,214	$226,176	$13,311	$1,222	$775	$20,202	$186,665		748,565				Arizona.
Arkansas	4,212,161	4,125,968			42,120		6,510	23,019	14,544	84,243	914,039	543,369	[5] 2,670,510		Arkansas.
California	10,489,068	9,026,648	5,729,508	3,297,140	330,908	34,681	48,540	327,085	721,206	2,098,197	4,195,436	4,195,435			California.
Colorado	1,835,386	1,691,618	1,382,742	308,876	2,138	1,985	30,256	15,765	93,624	166,652	834,367	834,367			Colorado.
Connecticut	7,992,755	6,075,244	4,630,608	1,444,636	12,675	10,386	92,137	1,160,031	642,282	618,158	7,374,597				Connecticut.
Delaware	1,023,440	825,115	602,287	222,828	5,298	1,225	7,680	183,259	863		1,023,440				Delaware.
Florida	4,959,129	4,893,564	3,601,701	1,291,863		5,291	25,329	4,279	30,666	297,548	3,496,186	1,165,395			Florida.
Georgia	4,568,209	4,491,505	3,705,564	785,941	20,199	5,619	38,427	5,859	6,600	164,825	4,403,384				Georgia.
Idaho	1,787,467	1,743,294	1,431,716	311,578	8,422	1,773	27,735	1,560	4,683	29,344	175,813	1,582,310			Idaho.
Illinois	17,087,209	15,938,407	12,249,403	3,689,004	84,549	19,770	90,140	373,679	580,664		9,623,138		7,428,636	[6] $35,435	Illinois.
Indiana	6,253,424	5,727,856	4,404,613	1,323,243	52,387	5,202	51,230	37,130	379,619	276,100	5,977,324				Indiana.
Iowa	11,919,350	11,048,755	9,849,983	1,198,772	6,511	5,871	84,166	38,134	735,913	417,177	10,962,849	392,225		[7] 147,099	Iowa.
Kansas	5,697,306	5,678,086							19,220	292,602	3,804,704	1,600,000			Kansas.
Kentucky	5,381,302	5,204,833	4,023,896	1,180,937		5,638	33,262	47,064	90,505	235,562	4,555,272	590,468			Kentucky.
Louisiana	4,523,634	4,456,325						67,152	157	50,000	4,473,634				Louisiana.
Maine	3,030,128	2,316,935	1,762,353	554,582	5,126	6,572	45,687	478,411	177,397	267,806	1,958,220		664,102	[8] 140,000	Maine.
Maryland	3,295,314	2,538,840	2,263,128	275,712	22,306	9,755	36,162	211,658	476,593	329,531	2,372,626			[9] 593,157	Maryland.
Massachusetts	7,117,725	4,125,779	2,927,100	1,198,679	24,310	8,768	9,283	1,888,676	1,060,909	1,392,066	4,995,116		730,543		Massachusetts.
Michigan	23,212,316	21,188,145	16,296,515	4,891,630	315,204	15,600	101,863	297,657	1,293,847	891,104	15,239,152	6,000,000	1,082,060		Michigan.
Minnesota	10,846,826	10,690,671	8,871,718	1,818,953	21,570	7,081	39,384	55,647	32,473		6,990,359		[10] 3,800,527	[6] 55,940	Minnesota.
Mississippi [2]	2,963,381									151,380	253,401	2,558,600			Mississippi [2]
Missouri	9,690,727	9,663,645							27,082	454,982	6,168,245		3,067,500		Missouri.
Montana [2]	1,549,487									86,914		1,429,260		[11] 33,313	Montana.[2]
Nebraska	4,289,969	4,028,067	3,411,760	616,307	8,660	3,236	58,672		191,334	105,927	1,239,342	2,891,798		[7] 52,902	Nebraska.
Nevada	296,881	292,172				475	4,000		234	15,524	107,385		173,972		Nevada.
New Hampshire	2,248,213	1,796,597				6,615	30,833	291,207	122,961	175,797	2,057,374			[7] 15,042	New Hampshire.
New Jersey	14,803,016	10,814,412	6,850,483	3,963,929	88,322	13,062	85,240	2,901,719	900,261	948,071	8,716,581	4,782,258		[12] 356,106	New Jersey.
New Mexico	756,763	723,234	640,505	82,729	5,648	568	11,144		16,169	74,692	449,726	224,862	[13] 7,483		New Mexico.
New York	38,293,313	34,040,850	24,411,272	9,629,578	147,714	59,210	216,279	3,146,192	683,068	1,415,965	27,360,992	5,806,006		[14] 3,710,350	New York.
North Carolina [2]	7,045,116									300,000	4,239,853		2,505,263		North Carolina.[2]
North Dakota	1,989,475	1,973,344	1,564,028	409,316		1,222	14,909			262,915	[15] 928,280	798,280			North Dakota.
Ohio	12,860,453	12,324,285						11,125	525,043	390,000	6,430,226	6,040,227			Ohio.
Oklahoma [2]	6,964,360									215,310	2,770,092	3,978,958			Oklahoma.[2]
Oregon	7,644,226	7,346,205	6,327,607	1,018,598		9,597	29,213	62,932	196,279	351,325	2,142,138	1,807,381	3,343,382		Oregon.
Pennsylvania	29,264,695	21,932,837	15,406,163	6,526,674	57,423	33,426	360,371	2,608,203	4,272,435	1,750,000	23,821,267		2,761,783	[6] 931,645	Pennsylvania.
Rhode Island	2,403,809	1,985,526	1,531,955	453,571	1,848	3,322	16,030	314,948	82,135	229,644	2,117,414	45,338		[16] 11,413	Rhode Island.
South Carolina	2,674,379	2,509,158	2,109,228	399,930	31,042	1,470	30,882		101,827	29,907	2,644,472				South Carolina.
South Dakota	3,150,657	3,064,539	2,629,191	435,348		964	27,750		57,404	63,013	1,575,329	1,512,315			South Dakota.
Tennessee [2]	4,288,420									107,368	2,090,526	2,090,526			Tennessee.[2]
Texas	20,418,696	19,678,457	16,240,125	3,438,332	205,932	15,918	71,160	37,439	409,790	796,572	11,521,233	8,100,891			Texas.
Utah [2]	838,300									130,000	372,900		335,400		Utah.[2]
Vermont	2,339,782	2,007,736	1,666,898	340,838		3,343	33,945	247,292	47,466		2,339,782				Vermont.
Virginia	6,145,296	5,662,036	4,794,734	867,302	8,164	8,121	72,902	52,367	341,706	260,000	5,885,296				Virginia.
Washington	7,547,382	6,962,851	5,451,794	1,511,057	59,766	12,647	90,606	407,958	13,554	346,714	4,561,223	2,231,487		[8] 407,958	Washington.
West Virginia	4,565,836	4,131,275	3,223,064	908,211	5,105	6,095	48,640	138,679	236,042	215,621	1,210,460		3,106,667	[7] 33,088	West Virginia.
Wisconsin	11,780,703	11,382,876	9,072,253	2,310,623	20,180	17,653	107,815	71,703	180,476	650,000	6,199,689	4,931,014			Wisconsin.
Wyoming [2]	647,200										647,200				Wyoming.[2]
District of Columbia	665,914	158,672	135,675	22,997		1,009	2,107	191,763	312,363	86,423				[17] 579,491	District of Columbia.
Detailed totals [2]	319,810,899	284,792,752	([18])	([18])	1,606,838	344,392	2,081,064	15,719,794	15,266,059						Detailed totals.[2]
Grand total	347,843,543									17,402,798	223,292,969	66,861,364	33,183,473	7,102,939	Grand total.

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[1] Financial data only on this table. For number of registrations, etc., see preceding page.

[2] Eight states thus noted do not report complete details, and receipts are not included in "detail totals."

[3] States for which no figures are shown make appropriations for administration.

[4] Payments on State highway bonds, except as noted.

[5] Includes payments on county bonds of $1,971,291.

[6] For State general funds.

[7] For State motor vehicle fee refunds.

[8] For State highway motor patrol.

[9] For Baltimore city streets.

[10] County bond payments assumed by State.

[11] For auto theft fund.

[12] Includes $105,106 for new State highway office building and $251,000 for free bridge commission.

[13] Interest on notes outstanding.

[14] For New York City general fund.

[15] Includes special State bridge fund.

[16] For administration of fuel tax law.

[17] For traffic control signals, and streets, as appropriated by Congress.

[18] Only 34 States report detail of motor car receipts as follows: Passenger car and bus receipts $189,499,784, and truck and tractor receipts $56,955,890, making a total of $246,455,674.

GASOLINE TAXES, 1929

Total tax earned on motor vehicle fuel, etc., refunds, disposition of fund, and gallons taxed

[From reports of State authorities]

State	Gross tax assessed prior to deduction of refunds	Exemption refunds: (Deducted from gross tax)	Total tax earnings on fuel for motor vehicles [1]	Other receipts under tax law (licenses)	Grand total earnings (tax and other receipts)	Disposition of grand total earnings					Tax rates, 1929			Net gallons of gasoline taxed, and used by motor vehicles
						Collection costs [2]	Construction and maintenance on rural roads		State and county road bond payments [3]	For miscellaneous purposes	Cents per gallon		Date of rate change	
							State highways	Local roads			Jan. 1	Dec. 31		
Alabama	$7,105,009		$7,105,009		$7,105,009	$37,853	$2,367,023	$3,522,447	$1,177,686		4	4		[4] 178,162,903
Arizona	2,886,603	$326,772	2,559,831	$8	2,559,839	(5)	1,599,894	959,945			4	4		63,995,783
Arkansas	6,681,029	(6)	6,681,029		6,681,029	33,405	4,195,686	420,905	[7] 2,031,033		5	5		133,620,566
California	37,600,108	3,408,021	34,192,087		34,192,087	8,566	23,181,265	10,960,668		[8] $41,588	3	3		1,139,736,244
Colorado	5,745,061	527,582	5,217,479	585	5,218,064	[9] 46,915	3,619,804	1,435,478		[10] 115,867	3	4	May 1	141,466,891
Connecticut	4,047,092		4,047,092	50,083	4,097,175		4,097,175				2	2		202,354,590
Delaware	980,559	44,612	935,947		935,947		935,947				3	3		31,198,248
Florida	12,203,056		12,203,056	28,280	12,231,336	16,477	5,666,821	1,544,146		[11] 5,003,892	5	6	July 1	223,373,467
Georgia	10,224,108		10,224,108		10,224,108	4,200	6,567,409	2,195,045		[12] 1,457,454	4	6	Sept. 1	219,609,473
Idaho	2,204,977	258,997	1,945,980	379	1,946,359	13,813	1,932,546				4	4		48,658,984
Illinois [13]	11,659,778		11,659,778		11,659,778	[14] 25,000	[15] 7,756,519	[15] 3,878,259			0	3	Aug. 1	[13] 388,659,266
Indiana	16,429,211	818,960	15,610,251	289	15,610,540	33,102	11,683,078	2,920,770		[16] 973,590	3	4	Apr. 1	410,936,759
Iowa	10,068,831	713,046	9,355,785		9,355,785	31,728	4,611,780	4,712,277			3	3		311,859,516
Kansas	9,733,375	1,562,170	8,171,205		8,171,205	(17)	6,371,205	1,800,000			2	3	Apr. 1	288,716,546
Kentucky	7,742,564		7,742,564		7,742,564	25,777	7,716,787				5	5		154,717,831
Louisiana	6,979,699	1,048	6,978,651		6,978,651	(18)	5,255,782		1,722,869		2	4	Jan. 3	176,645,631
Maine	3,841,476	132,794	3,708,682		3,708,682	26,052	2,301,644	1,380,986			4	4		91,610,422
Maryland	6,532,418	235,250	6,297,168		6,297,168	2,500	5,035,734			[19] 1,258,934	4	4		157,429,197
Massachusetts	9,888,730	129,914	9,758,816		9,758,816	20,000	6,770,807	2,968,009			2	2		487,940,778
Michigan	23,558,077	2,249,068	21,309,009	3,920	21,312,929	40,286	11,058,317	6,810,406	3,000,000	[20] 403,920	3	3		710,300,302
Minnesota	9,631,151	739,026	8,892,125		8,892,125		6,283,903	2,608,222			2	3	May 1	338,631,771
Mississippi	7,045,120		7,045,120	131,006	7,176,126	5,900	2,736,601	4,223,532		[21] 210,093	5	5		[22] 140,902,401
Missouri	7,866,605	185,933	7,680,672		7,680,672	54,940	7,625,732				2	2		384,033,575
Montana	3,767,737	965,720	2,802,017		2,802,017	13,584	2,788,433				3	5	Apr. 1	57,514,249
Nebraska	7,860,046	60,567	7,799,479		7,799,479	7,500	5,982,429	1,809,550			2	4	Mar. 29	208,869,358
Nevada	712,507	60,206	652,301		652,301		599,377	52,924			4	4		16,307,535
New Hampshire	2,332,906	65,854	2,267,052		2,267,052		1,700,289		566,763		4	4		56,676,294
New Jersey	9,961,276	(23)	9,961,276	34,828	9,996,104	20,040	9,886,064			[24] 90,000	2	2		498,063,808
New Mexico	2,273,966		2,273,966	15,801	2,289,767	45,795	1,752,722		491,250		5	5		45,479,332
New York [25]	19,488,422	433,248	19,055,174	32,218	19,087,392	(26)	14,278,044	3,807,478		[27] 1,001,870	0	2	May 1	[25] 962,601,285
North Carolina	12,506,185	499,801	12,006,384		12,006,384	9,000	7,702,647		4,294,737		4	5	Apr. 1	260,210,528
North Dakota	2,968,370	1,169,144	1,799,226	1,876	1,801,102	25,000	1,396,102	380,000			2	3	July 1	71,591,708
Ohio	35,741,107	1,658,919	34,082,188		34,082,188		21,316,968	6,567,787		[28] 6,197,433	3	4	Apr. 17	910,154,885
Oklahoma	10,870,642	29,033	10,841,609		10,841,609		7,798,066	3,043,543			3	4	June 22	314,388,292
Oregon	4,888,886	346,284	4,542,602		4,542,602	8,851	4,533,751				3	[29] 3		[30] 152,090,900
Pennsylvania	35,887,503	140,189	35,747,314	10,502	35,757,816	145,825	27,061,099	5,217,705	3,333,187		3	4	July 1	1,047,914,175
Rhode Island	1,556,538	13,779	1,542,759	3,202	1,545,961		1,160,271		385,690		2	2		77,826,879
South Carolina	6,895,951	29,343	6,866,608	4,468	6,871,076		5,725,897	1,145,179			5	6	Mar. 16	118,038,130
South Dakota	4,680,153	1,134,388	3,545,765		3,545,765	10,500	2,585,894		845,416	[8] 103,955	4	4		88,644,138
Tennessee	9,290,853		9,290,853		9,290,853	46,454	4,936,989	1,719,329	[31] 2,588,081		3	5	Feb. 17	194,497,225
Texas	23,092,702	775,208	22,317,494		22,317,494		16,738,121			[32] 5,579,373	2	4	July 16	761,421,692
Utah	1,979,226		1,979,226	384	1,979,610	4,150	1,536,960		438,500		3½	3½		56,546,967
Vermont	1,703,091		1,703,091		1,703,091		1,703,091				3	4	Apr. 1	43,990,554
Virginia	10,419,421	524,480	9,894,941		9,894,941	(33)	6,596,627	3,298,314			5	5		197,898,821
Washington	6,461,106	518,067	5,943,039		5,943,039	(34)	4,663,362	1,279,677			2	3	July 1	233,333,570
West Virginia	5,061,572	195,380	4,866,192	7,106	4,873,298		2,376,725		2,496,573		4	4		121,654,788
Wisconsin	7,867,899	382,860	7,485,039		7,485,039	10,899	2,752,212	4,183,051		[16] 538,877	2	2		374,251,957
Wyoming	1,296,299		1,296,299		1,296,299	4,066	1,024,157	268,076			3	4	Apr. 1	34,242,816
District of Columbia	1,437,754	9,573	1,428,181		1,428,181					[35] 1,428,181	2	2		71,409,032
Total			431,311,519	324,935	431,636,454	778,178	297,967,756	85,113,708	23,371,785	24,405,027	Aver. rate..3.22			,180,062

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[1] This is the net gasoline tax earned after deduction of refunds for exemptions according to law.

[2] Many States pay collections costs from other State funds, and when the amounts are reported they are noted.

[3] Payments for State highway bonds except as noted.

[4] Includes 1,075,385 gallons at 2-cent rate under old law but not previously reported.

[5] Paid from State budget, $14,433.

[6] No refunds reported this year.

[7] Consists of $514,439 for State bonds and $1,516,694 for county bonds.

[8] Reserve for refunds.

[9] Includes all expenses of State inspector of oils.

[10] For town and city streets.

[11] Consists of $2,758,354 for city streets, $2,233,735 for schools and permanent buildings, and $11,803 for refund reserve.

[12] For public schools.

[13] For last 5 months of year.

[14] Estimate from 1927 data.

[15] Funds held by courts pending decision.

[16] For city streets.

[17] From State general fund, $15,000.

[18] State appropriation of $7,500.

[19] For Baltimore city streets and grade crossings.

[20] Consists of $400,000 for city streets and $3,920 (from dealers' licenses) to State general fund.

[21] Special additional tax of 2 and 3 cents in Hancock and Harrison Counties for sea wall to protect road.

[22] Gallons taxed for highway purposes and also reported separately as taxed for sea wall included only once.

[23] Exemption claimed upon purchase.

[24] For State department of commerce and navigation.

[25] For last 8 months of year.

[26] Charged to State general fund, $24,504.

[27] Consists of $50,000 for reserve for refunds and $951,870 for New York City.

[28] Consists of $75,000 for rotary fund for refunds and $6,122,433 for municipal street

[29] Tax increased to 4 cents on Jan. 1, 1930.

[30] Includes 4,025,392 gallons of distillate taxed at 2½ cents.

[31] Consists of $652,833 for State bond payments and $1,935,249 for county bonds

[32] Free school fund.

[33] State appropriation of $11,088.

[34] Paid from motor-vehicle fund, $7,000.

[35] For repair and improvement of Washington streets.

[36] Approximate grand total of gallons of gasoline consumed by motor vehicles allowing for estimated gallon used in Illinois and New York during early months in year when no tax was assessed.

SOIL SAMPLING WITH A COMPRESSED AIR UNIT

By H. F. BLANEY, Associate Irrigation Engineer, and C. A. TAYLOR, Assistant Irrigation Engineer, Division of Agricultural Engineering, Bureau of Public Roads

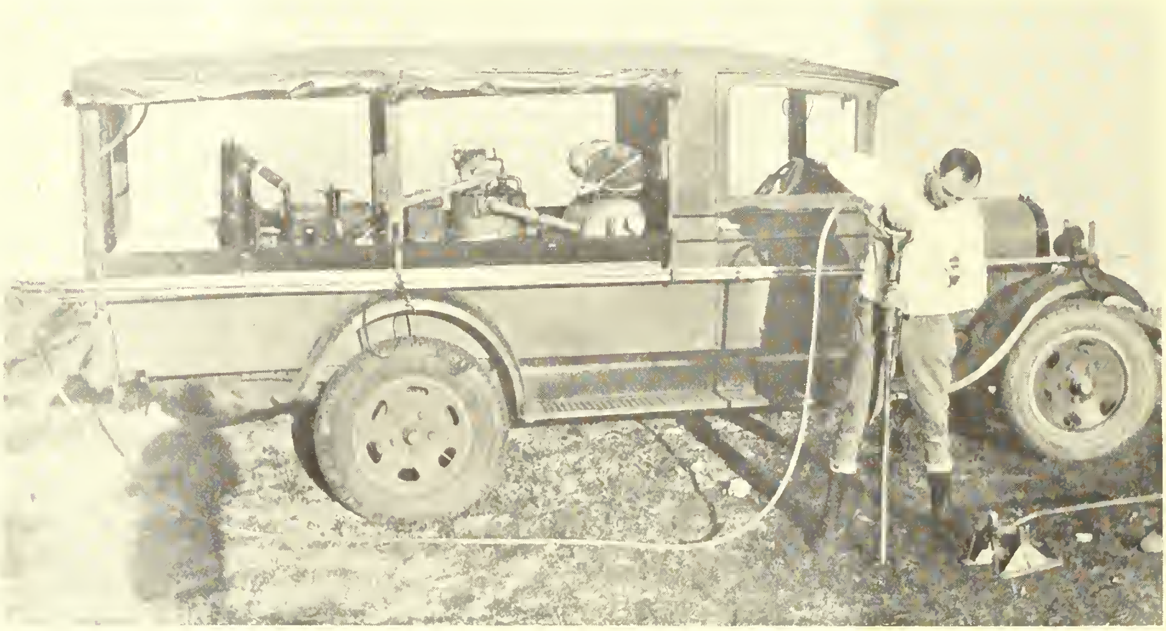

Figure 1.—Compressor Unit Mounted on Truck

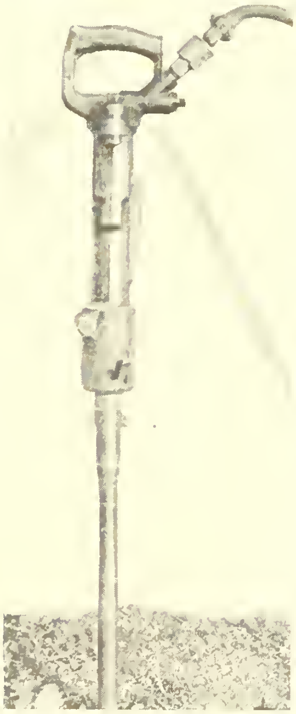

Figure 2.—Air Hammer

In making investigations regarding rainfall penetration in southern California, the authors[1] are conducting soil moisture studies in which soil samples are taken at 1-foot intervals to a depth of 18 feet, using the improved soil tube.[2] The success of this work depends, to a considerable extent, upon the speed with which a large number of samples can be obtained, and since the physical effort required may become the limiting factor in obtaining samples at considerable depths, a compressed-air unit has been adapted for use in driving the tubes in order that as many samples as desired can be taken quickly and easily. A description of the apparatus is presented as it may be of interest to investigators in several fields, such as that of highway engineering.

The apparatus consists of a compressor unit mounted on a truck, a light air hammer, and a 100-foot length of hose. Figures 1 and 2 show the mounting of the apparatus and the manner in which soil tubes are driven. The compressor unit consists of a 2-cylinder air compressor coupled directly to a 4-cylinder gasoline engine with a self-starter. The air receiver and gasoline tank are mounted in front of the compressor. The complete unit is mounted on a channel iron frame and bolted to one side of the truck floor, thus leaving one-half of the truck body space available for carrying other equipment. A reel with 100 feet of ½-inch rubber hose is mounted at the back of the truck, and special hose couplings permit connections to be quickly made to the hammer and compressor. An air pressure control automatically maintains any desired pressure up to 150 pounds per square inch and the displacement of the compressor is 59 cubic feet per minute at 800 revolutions per minute.

The hammer is of the clay-digger type, capable, when working under an air pressure of 100 pounds per square inch, of delivering 2,250 blows per minute, each blow striking with a force of 16 foot-pounds. Extending down into the soil tube is a 6-inch guide rod, the shoulder of which may be seen resting against the end of the soil tube in Figure 2. The trigger grip gives the operator ready control of the hammer, and by properly cramping the guide rod in the end of the soil tube, very little vibration is transmitted to the arms of the operator. The jack[3] used for pulling the soil tubes is shown on the ground in Figure 1.

The set-up for sampling is quickly made, as it is only necessary to reel the hose out to the desired location and snap it onto the compressor. This unit has been in use since October, 1929, and has proved very satisfactory. It is estimated that the time of sampling has been cut to one-third of that required for handwork.

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[1] Under the supervision of W. W. McLaughlin, Associate Chief of the Division of Agricultural Engineering, Bureau of Public Roads, United States Department of Agriculture, in cooperation with the State of California, Department of Public Works.

[2] "An Improved Soil-sampling Tube," by F. J. Veihmeyer, Soil Science, Vol XXVII, No. 2 (February, 1929).

[3] "An Efficient Soil Tube Jack," by C. A. Taylor and H. F. Blaney, Soil Science, Vol. XXVII, No. 5 (May, 1929).

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Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
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Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

MARCH 31,1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	2,120.6	$ 1,947,423.11	$ 957,297 62	69.8	21.0	90.8	$ 510,075.83	$ 255,037.91	21.7	1.1	22.8	$ 3,202,594.73	Alabama
Arizona	818.3	3,532,598.06	2,651,919.56	124.2	133.8	258.0	215,374.91	187,121.20	4.6	19.2	23.8	1,849,258.88	Arizona
Arkansas	1,737.8	5,473,611.07	2,551,362.89	143.8	31.9	175.7	1,324,370.00	565,075.03	54.2	18.8	73.1	1,333,302.74	Arkansas
California	1,884.6	6,745,856.53	2,883,289.64	167.9	8.9	176.8	1,277,090.21	267,727.02	6.7	16.0	22.7	1,467,821.23	California
Colorado	1,179.6	4,529,095.13	2,358,012.11	178.1	38.7	216.8	571,883.75	318,322.63	32.3		32.3	2,145,030.42	Colorado
Connecticut	240.3	1,095,890.67	455,428.80	6.5		6.5	1,336,849.84	667,777.59	7.1		7.1	518,817.96	Connecticut
Delaware	249.3	601,315.95	222,817.99	16.9		16.6	374,914.90	172,500.95	15.9		15.9	239,106.44	Delaware
Florida	467.2	5,002,294.59	2,269,116.74	102.0	5.5	107.5	953,391.19	436,618.46	31.4		31.4	1,148,290.48	Florida
Georgia	2,721.2	1,131,814.96	536,760.13	38.3	8.5	46.8	358,129.82	178,524.78	20.4	.3	20.7	3,860,711.80	Georgia
Idaho	1,193.9	713,108.23	430,934.62	55.7	27.8	83.5	242,133.15	145,452.71	24.3		24.3	1,311,447.00	Idaho
Illinois	2,034.5	15,137,362.22	6,676,755.85	430.1		430.1	609,858.67	304,929.33	20.7		20.7	5,742,647.49	Illinois
Indiana	1,344.0	7,509,467.02	3,525,586.95	233.1		233.1	374,724.44	178,170.00	11.9		11.9	1,944,445.43	Indiana
Iowa	3,130.0	818,396.20	343,538.93	15.1	18.5	33.6	4,594,049.99	2,034,098.01	30.6	138.9	159.5	37,833.61	Iowa
Kansas	2,717.6	4,755,346.35	2,197,784.31	294.0	17.5	311.5	1,478,303.18	727,360.10	75.9	8.3	84.2	1,514,965.75	Kansas
Kentucky	1,397.8	4,238,953.63	2,013,037.35	235.5	20.2	255.7						1,858,480.70	Kentucky
Louisiana	1,355.7	4,070,418.63	1,988,555.96	128.1	12.5	138.8	626,770.55	307,613.70	10.1	1.8	11.9	1,473,105.97	Louisiana
Maine	520.0	1,905,021.13	736,937.56	47.5		47.5	87,912.01	43,956.00	5.6		5.6	1,530,800.42	Maine
Maryland	630.7	1,428,486.60	598,781.99	58.0		58.0	204,641.50	92,829.63	7.6	2.1	9.7	542,274.28	Maryland
Massachusetts	660.2	2,357,902.66	733,497.62	31.0		31.0	575,151.16	183,051.50	8.0	2.6	10.6	2,103,642.80	Massachusetts
Michigan	1,605.1	9,391,839.03	4,001,321.45	209.6	30.5	240.1	421,348.73	204,445.00	17.8		17.8	2,096,433.80	Michigan
Minnesota	4,107.5	5,684,979.57	1,554,300.00	144.9	85.3	230.2	5,328,379.60	1,190,475.61	77.0	161.4	238.4	147,703.39	Minnesota
Mississippi	1,810.5	1,846,209.28	716,929.45	55.6	7.7	73.3						2,647,800.78	Mississippi
Missouri	2,435.5	5,373,345.10	2,046,191.17	74.1	66.7	140.8	7,012,636.09	2,309,410.71	95.3	73.9	169.2	195,638.06	Missouri
Montana	1,722.2	7,543,696.71	4,393,686.42	519.4	12.8	532.2	970,567.46	527,822.26	63.2	50.8	114.0	2,953,296.35	Montana
Nebraska	3,596.3	7,290,479.05	3,471,451.51	330.5	148.1	478.6	1,271,876.37	531,698.22	29.4	39.4	68.9	1,707,151.01	Nebraska
Nevada	1,124.1	1,274,254.02	1,122,892.45	120.4	96.8	217.2	301,187.58	267,605.15		32.7	32.7	565,156.33	Nevada
New Hampshire	350.6	414,924.02	111,495.00	5.4	2.1	7.5	183,425.63	71,206.14	2.1		2.1	377,976.34	New Hampshire
New Jersey	500.2	4,136,550.77	852,015.00	56.8		56.8						1,091,319.09	New Jersey
New Mexico	1,935.0	3,222,209.17	2,043,523.55	186.5	2.1	188.6	814,414.64	571,458.47	18.4	18.2	36.6	711,851.51	New Mexico
New York	2,458.2	18,480,217.76	3,787,505.00	253.2		253.2	5,596,400.00	989,550.00	65.7		65.7	6,307,935.05	New York
North Carolina	1,755.2	1,519,021.81	748,811.25	92.0	22.3	114.3	1,508 490.24	745,040.21	76.1		79.1	2,112,719.58	North Carolina
North Dakota	4,175.7	1,196,709.89	482,732.47	205.4	101.6	307.0	1,796,183.97	916,867.41	313.1	165.8	479.9	1,124,324.24	North Dakota
Ohio	2,177.8	12,929,559.21	3,750,225.17	206.9	18.8	225.7	6,881,324.21	2,418,079.39	135.5	20.3	155.8	1,662,333.36	Ohio
Oklahoma	1,873.8	2,684,035.66	1,198,801.72	80.4	29.7	110.1	2,478,599.42	1,070,123.01	61.1	43.2	104.3	617,362.08	Oklahoma
Oregon	1,147.7	2,809,226.36	1,687,826.48	176.9	56.2	233.1	2,254,451.65	1,263,658.38	50.9	29.2	80.1	255,940.38	Oregon
Pennsylvania	2,265.2	14,653,228.83	3,494,552.43	212.9	14.1	227.0	4,275,514.21	1,124,861.25	60.0		60.0	2,051,894.90	Pennsylvania
Rhode Island	184.8	1,381,818.85	395,698.58	18.5		18.5						617,574.62	Rhode Island
South Carolina	1,896.7	2,999,920.73	1,035,176.18	86.6	22.3	108.9	1,579,062.09	571,970.84	41.0	32.9	73.9	840,058.19	South Carolina
South Dakota	3,430.7	4,057,731.42	2,162,173.18	423.7	116.8	540.5	602,425.30	308,794.28	46.7	47.8	94.5	783,348.50	South Dakota
Tennessee	1,243.3	1,871,857.29	817,145.16	74.8		74.8	685,577.23	342,788.61	19.0	12.5	31.5	2,139,265.12	Tennessee
Texas	6,745.0	10,826,830.66	4,394,669.44	311.6	89.6	401.2	2,527,554.33	1,006,611.67	110.6	23.9	134.5	3,844,092.67	Texas
Utah	988.9	589,171.08	381,932.01	25.8		25.8	404,693.19	296,379.18	21.8	8.1	29.9	745,883.39	Utah
Vermont	257.8	1,009,330.13	299,380.24	20.1		20.1	1,343,211.60	259,344.01	22.4	2.9	25.0	91,873.19	Vermont
Virginia	1,413.9	3,336,503.10	1,562,568.09	145.1	15.6	160.7	1,249,474.85	600,904.46	35.1	7.7	42.9	389,648.61	Virginia
Washington	914.4	3,238,771.51	1,215,000.00	64.9	40.2	105.1	742,854.61	324,200.00	24.3		24.3	1,036,961.29	Washington
West Virginia	712.9	3,170,534.29	1,207,156.33	75.4	12.5	87.9	735,889.87	350,767.91	14.1	6.2	20.3	455,471.79	West Virginia
Wisconsin	2,259.1	3,194,[illegible]16.98	1,399,185.13	121.0	1.7	122.7	664,293.71	270,000.00	18.8		18.9	1,627,965.88	Wisconsin
Wyoming	1,757.1	1,333,[illegible]30.81	862,873.64	118.6	24.3	142.9	34,843.01	22,874.43	4.7		4.7	929,005.82	Wyoming
Hawaii	39.5	964,[illegible]49.48	384,686.43	23.2		23.2						1,438,289.15	Hawaii
TOTALS	83,298.0	211,430,846.31	85,913,322.25	9,823.6	1,362.6	8,186.1	67,331,304.68	[illegible],523,103.05	1,813.1	985.8	2,798.9	75,489,846.90	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE: 1930

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 3 ▽ MAY, 1930

A FEDERAL-AID ROAD IN PENNSYLVANIA

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 11, NO. 3 **MAY, 1930** **R. E. ROYALL, Editor**

TABLE OF CONTENTS

Page

A Technical Basis for Apportioning Motor Vehicle Taxes - - - - - - 41

A Method of Analysis of Data on Frost Occurrence for Use in Highway Design - - - 51

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A TECHNICAL BASIS FOR APPORTIONING MOTOR VEHICLE TAXES[1]

By CHARLES F. MARVIN, Jr., Mechanical Engineer, Heat and Power Division, U. S. Bureau of Standards

THE rapid increase in motor transportation which has occurred in the past 30 years would not have been possible without the highway improvements and extensions which have accompanied the development of the motor car.

These highway improvements and extensions threw upon the State and local governments a great additional financial burden which, with their existing revenue systems, they were ill-prepared to assume. Since no part of this increase in highway expenditures could be attributed to any factor other than the new demands of automobile traffic, it seemed equitable and expedient to impose special taxes on motor vehicle owners to meet the cost, in part at least, of the special service provided. After years of application, State registration or license fees and fuel taxes have come to be accepted as just and logical forms of taxation.

However, at the present time there is no generally accepted basis for determining the amounts of fees for the various classes of vehicles or the units within a class. Neither is there any agreement as to how the total tax levied by a State should be apportioned between the fuel tax and registration fees. Each State has selected its own method of levying fees with the result that at the present time vehicles are rated according to chassis weight, unladen weight, capacity, gross weight, horsepower, piston displacement, value, tire width, or tire type as well as on several different combinations of two or more of these characteristics. In many cases there is no definite relation between the characteristic used as a base and the justification for taxing or the amount of fee charged. Some States rate passenger cars and trucks on the same basis while others do not. Several States have sacrificed simplicity of administration in an effort to obtain more equitable fees, while other States, apparently aiming at simplicity, have prescribed fees which seem inconsistent in their distribution of the tax burden among vehicles of different characteristics. All of the States and the District of Columbia impose fuel taxes of from 2 to 6 cents a gallon, but there is no consistent relation between the amounts of the fuel taxes and the amounts of registration fees.

Several years ago the motor vehicle and highway departments of the State of Connecticut requested the Bureau of Standards to undertake a study of current systems for taxing motor vehicles with a view to eliminating some discrepancies in the Connecticut system and, if possible, to find a more logical and equitable basis for such taxation.

The present paper is a revision of the report which was submitted to the State of Connecticut, with the material of general interest rearranged to emphasize the principles involved and with those portions applying only to Connecticut omitted.

It was not within the scope of the investigation to recommend a total revenue to be collected from motor vehicle owners, or to fix the relation between such a total and the total highway expenditures. Each State must determine for itself how much it wishes to spend on highways and what percentage of this amount should be financed by motor vehicle taxes, taking into account general social benefits and benefits to property which may be expressed by general appropriations and property taxes. The purpose of the bureau's investigation was to recommend a simple and equitable method of distributing a total tax levy, determined by the State, among the various classes of vehicles and the units within a class as nearly as possible according to the amount and character of each vehicle's use of the public streets and roads. In this distribution no attempt was made to devise methods of collecting taxes on a basis of earning capacity or ability to pay. No discrimination was made as between privately operated vehicles and those operated for hire or by dealers. If such discriminations are thought advisable, they may be made in the form of a separate or an additional tax.

NECESSARY ELEMENTS OF A SATISFACTORY TAX SYSTEM

Equity.—A satisfactory tax system must first of all be equitable, that is, it must distribute the cost among the different vehicles in such a manner that each vehicle is assessed according to the justification for taxing such vehicle. The greatest and most evident justification for taxing is to defray the expenses incurred by the State due to the operation of the motor vehicles, and the first approximation to an equitable system would be one which would tax each vehicle in proportion as that vehicle is responsible for these expenses. It is also desirable in the interests of public utility, and convenience, that taxes should be so imposed that they penalize wherever possible those vehicles which are inefficient in their use of the highways or the nation's fuel resources, and thus encourage a relative increase in vehicles which bring about improvement and progress in highway transportation.

The expenses incurred by the State due to the operation of a given vehicle and the desirability of the vehicle as a unit of transportation will depend, in varying degree, on such things as the weight, size, speed, capacity and tire equipment of the vehicle; the number of miles it travels during the taxable period; the type of road over which it operates; the manner in which it is cared for and operated; the amount of regulation it requires; and the character of the service which it renders.

Simplicity.—It is highly desirable that the process of administering taxes be as simple as possible. Unfortunately, the exactly equitable rating, depending as it does on so many inter-dependent factors of varying importance, would be extremely complex. Even if it could be exactly determined, it would be impractical to administer. All that can be hoped for is a compromise rating which will be equitable in the most important respects and sufficiently simple to be cheaply and efficiently administered.

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[1] Publication approved by the Director of the Bureau of Standards of the United States Department of Commerce.

Undoubtedly the major factors to be considered in taxing a vehicle are (1) the weight imposed upon the road surface by the vehicle and its load, (2) the number of miles it travels during the taxable period, and (3) the type of tire with which the vehicle is equipped. A system which attempts to include more factors than these becomes very unwieldy while one which neglects any one of the three does so at a serious loss of equity.

Adaptability.—The ideal tax system would be equally applicable to the taxing of all of the various classes of vehicles, readily adaptable to the requirements of any State, and capable of being easily modified to suit changing conditions in the same State, without alteration of the basic method of rating vehicles or the system of collecting fees.

The three factors, weight, miles traveled, and tire type, may well be used as bases for rating all vehicles and for determining fees in all States for some years to come. However, it will not be possible to use the same amounts or ranges of fees everywhere and for all time. Costs of highway construction as well as the types of road and the total mileages vary from State to State and from time to time. Vehicles in the different weight groups are distributed differently throughout the various sections of the country and the damage caused by vehicles of a certain class is not necessarily the same in one State as in another. For these reasons any general method for determining taxes must permit a wide range of tax schedules.

THE RECOMMENDED SYSTEM

The vehicle characteristic of most importance so far as the road is concerned is not the rated horsepower or the rated carrying capacity so often used in determining fees, but the gross weight of the loaded vehicle. It is gross weight which is actually carried by the roadway, gross weight which largely determines the amount of highway damage, and gross weight for which new bridges and highways must be designed. In addition, gross weight is the only characteristic common to all of the various types of vehicles which is, for each type, closely related to the use and consequent deterioration of the road. These facts combine to make gross weight the best single measure of the justification for taxing.

Since gross weight bears a definite relation to the justification for taxing, it should also be definitely related to the amount of tax by some formula which would insure a gradual and consistent variation in tax throughout the whole range of gross weights.

The general formula.—Almost any relation between tax and gross weight which is likely to be desired in practice can be represented algebraically with a fair degree of approximation by the following formula: $\text{Tax} = A + B \times (\text{gross weight}) + C \times (\text{gross weight})^2$ in which the tax is in dollars and gross weight is in tons.

Certain familiar types of tax schedule are represented by the three terms of the formula. Thus, the first term imposes a flat rate of A dollars. The second term, $B \times (\text{gross weight})$, levies a constant rate of B dollars a ton, while the last term, $C \times (\text{gross weight})^2$, is in effect a surtax which imposes an increasing rate per ton, and therefore affects most noticeably the heavier vehicles. By the proper evaluation of A, B, and C, the characteristics of these three distinct types of schedules may be blended into a single schedule in any proportion desired. The wide diversity of schedules which can be obtained is indicated by the examples given in Figure 1. Still other types can be obtained by using negative signs.

Three sets of corresponding values of tax and gross weight which will indicate the general trend desired in the tax-gross weight relation will serve to fix the constants A, B, and C. Thus, if it is decided, for example, that the total annual tax (registration fee plus annual fuel tax) for the average vehicle of 10 tons gross weight should be $200, as compared with $75 for a 5-ton vehicle and $15 for a 1-ton vehicle, then the formula becomes:

$\text{Total annual tax} = 5.55 + 8.34 \times (\text{gross weight}) + 1.11 \times (\text{gross weight})^2$* and the total annual taxes for average vehicles of other weights will be as shown by the heavy line in Figure 2. The numerical values given above and appearing in Figure 2 and Table 1 were chosen arbitrarily merely for illustration. For actual use they should be determined independently by the different States and changed from time to time to meet existing conditions.

The fuel tax.—The dotted lines in Figure 2 show how fuel taxes of different amounts affect vehicles of different weights traveling the same number of miles per year, in this case 6,000. It is obvious that a fuel tax alone can not impose the rapidly rising schedule of taxes generally required. The fuel tax, however, possesses several advantages over registration fees, chief of which is that it automatically taxes a vehicle directly according to the number of miles traveled. In addition, it is effective in distributing taxes in the proper directions, although not always in the desired amounts, according to other justifications for taxing.

Thus it brings in more revenue per mile from the heavy truck than it does from the light passenger car, and taxes the fully loaded vehicle more than the empty one.

The increase in the price of fuel, due to the tax, encourages greater economy in the design and operation of vehicles and penalizes the owner who operates a car which is wasteful of the Nation's fuel resources. Thus, the vehicle which is overweight, overpowered, which has an inefficient engine, an improperly adjusted carburetor, or excessive friction in any of its parts pays a higher tax per mile than the well-designed and well-cared-for vehicle of the same capacity.

A fuel tax brings in revenue from visiting cars from other States which would otherwise use the highways tax free.

A car which continuously maintains a high speed through the open country will burn more gasoline and therefore pay a somewhat higher tax than a similar car which is driven over the same roads at a moderate speed.

Vehicles which habitually operate in crowded districts where highway "rent" should be high and where expenses of regulating traffic are greatest will, on account of their frequent stops and starts, burn more fuel and therefore pay a higher tax per mile than similar vehicles operating in the open country.

These advantages, together with the fact that it is conveniently paid and simple to collect, have made the fuel tax an increasingly acceptable form of taxation.

To fully utilize the many advantages of the fuel tax, it should constitute the greatest proportion of the total tax which can be charged without overtaxing any class of vehicles. Usually, the most equitable fuel tax will be determined solely by the total annual

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* See Appendix A.

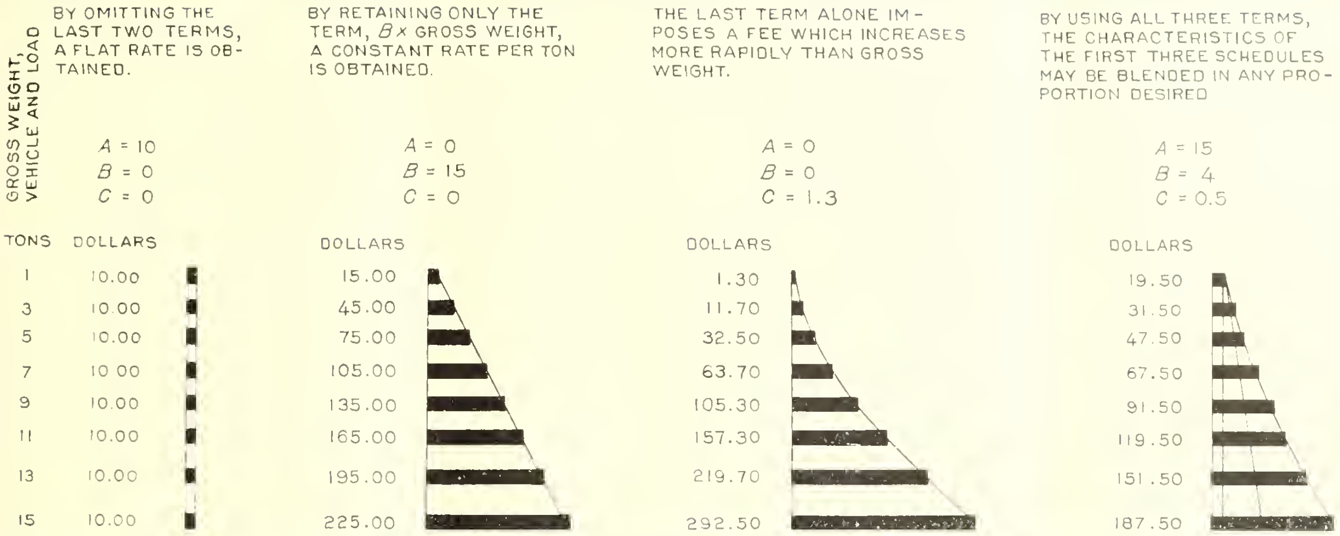

Figure 1.—Examples of the Various Types of Tax Schedules Obtainable by Varying the Constants in the General Formula: Tax = $A + B \times$ Gross Weight + $C \times$ Gross Weight^2

Table 1.—*Registration fees for pneumatic-tired vehicles according to formulas plotted in Figure 2, assuming a gasoline tax of 6 cents*

Gross weight of vehicle in tons	Fee in dollars	Gross weight of vehicle in tons	Fee in dollars
Up to 2	$1.00	6 to 7	$36.00
2 to 3	3.50	7 to 8	50.00
3 to 4	8.50	8 to 9	66.00
4 to 5	15.50	9 to 10	84.00
5 to 6	25.00	10 to 11	104.00

tax desired on the lightest class of cars. This is illustrated by the 6-cent fuel tax in Figure 2. If the total tax for the one-ton cars had been assumed as $10.00 instead of $15.00, the most equitable fuel tax would have been 4 cents instead of 6 cents per gallon.

In terms of the formula, the 6-cent tax becomes: Annual fuel tax = $3.00 + 11.30 \times$ (gross weight).*

Registration fees.—There are some very important cases in which the justification for taxing is not measured exactly by the fuel consumption, and a fuel tax is therefore inadequate or ineffective.

A fuel tax does not provide a means of collecting from the heavier vehicles their share of highway costs. It can not penalize for the extreme width usually found in heavy trucks and busses. Also the type of tire used has only a small effect on fuel consumption but a very decided effect on road cost. Electrically driven vehicles pay no fuel tax at all, and this type must be taxed entirely by some other method.

Vehicles must be registered for purposes of record and identification and it is convenient at the same time to collect registration fees of sufficient amount to compensate for the inadequacies of the fuel tax.

Subtracting the above formula for fuel tax from the one previously given for total tax gives a formula for registration fees, thus:

$$\begin{aligned}\text{Total annual tax} &= 5.55 + 8.34 \times (\text{gross weight}) + 1.11 \times (\text{gross weight})^2\\ \text{Annual fuel tax} &= 3.00 + 11.30 \times (\text{gross weight})\\ \hline \text{Registration fee} &= 2.55 - 2.96 \times (\text{gross weight}) + 1.11 \times (\text{gross weight})^2\end{aligned}$$

As shown in Figure 2 and Table 1, this equation gives very small registration fees for the light cars which are equitably taxed by the fuel tax alone, and is chiefly effective for the heavier trucks and busses for which the fuel tax is inadequate.

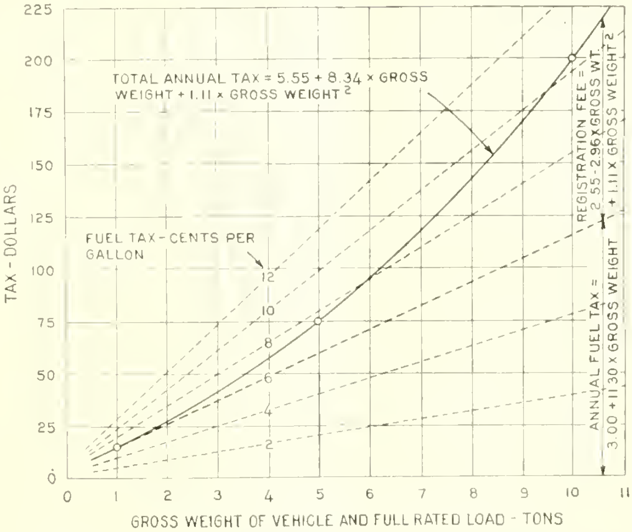

Figure 2.—Typical Tax Schedule Based on the Formula Tax = $A + B \times$ Gross Weight + $C \times$ Gross Weight^2

A still more rapidly rising schedule of registration fees may be secured for solid-tired vehicles by using a higher coefficient for the last term in the formula.

ADMINISTRATION

The tax schedule.—For administration purposes the tax schedule plotted and represented by formulas in Figure 2 would appear as in Table 1. In this table vehicles are divided into groups according to their gross weights and the registration fee for each group as determined by the formula is listed for convenient reference. It will be noticed that practically all of the passenger cars, except busses, fall in the first two groups, greatly simplifying the administration of fees for this most numerous class of vehicles.

Examples of the total annual taxes which will be paid by typical vehicles traveling 6,000 miles per year and paying the six-cent fuel tax and the registration fees in Table 1 are as follows:

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* See Appendix A.

Gross weight of car in tons	Total annual tax
2	$26.60
4	56.70
7	118.10
11	231.30

Fixing taxable gross weight.—The actual gross weight of a vehicle, of course, varies with the number of passenger or the quantity of goods carried and also depends on the character of the equipment and the amount of fuel, oil, and water in the tanks. It is therefore necessary for the State to designate what the taxable gross weight of any given vehicle will be.

Three general methods of fixing vehicle weight are used at the present time. These methods are as follows: (1) A table giving weights for the various makes and models of cars is adopted by the State as official; (2) a certificate of weight from a licensed public weigher is required by the State to accompany the application for registration; (3) the weight as declared by the applicant is accepted by the State.

The most popular and probably the most convenient method of fixing the taxable weight of passenger cars is by means of a table, prepared or adopted by the State, which gives the gross weight of each make and model of car and such other data as the year, body style, number of cylinders, etc., as would aid in identifying a car in the table. The gross weight given in the table should be the sum of the vehicle weight as furnished by the manufacturer plus a seating capacity times an estimated average weight per passenger of perhaps 125 or 150 pounds.

The vehicle weight without passengers of homemade cars, nonstock models, and cars not listed in the table should be determined by actual weighing, and the application for registration of such vehicles should be accompanied by a certificate of weight from a licensed public weigher. Any owner who wishes to have his car weighed and submit a certificate of weight in preference to accepting the vehicle weight for his car as listed in the State's tables should be permitted to do so.

The taxable gross weight, once determined for a car, should be printed on the registration card, and subsequent registrations of the same car could be made on the basis of the original weight, provided no changes had been made in weight or tire type.

The many variations in the weights of commercial vehicles, due particularly to the wide variety of body types, makes it impracticable to make a table of gross weights of commercial cars.

Several jurisdictions now require that certificates of vehicle weight accompany the applications for registration of such vehicles. To the vehicle weights thus fixed, the carrying capacities are added to give the taxable gross weights.

A simpler method would be to accept the declaration of the owner as to the weight of his empty vehicle and the weight of the greatest load he expects to carry, the sum of the two weights being the taxable gross weight. The declared capacity of a vehicle should not be greater than the maximum nor less than the minimum stated by the manufacturer and the gross weight should not exceed that specified in the laws regarding tire width and thickness. The vehicle weight, capacity and gross weight as declared should be painted on the vehicle where they can be readily seen. The strict enforcement of a law prohibiting the loading of a vehicle so that its gross weight exceeds the registered gross weight would discourage understatement of weight by the owner.

CONCLUSIONS

The general adoption of some simple formula such as that suggested would result in more logical and equitable relations between taxes and the justifications for taxing, and between the fuel tax and registration fees than exist at present in the schedules of most of the States. Moreover, it would provide for uniform basic methods of rating vehicles and collecting fees throughout the country, at the same time permitting the several States to select schedules suited to their individual requirements.

APPENDIX A

FIXING THE CONSTANTS IN THE FORMULA

As has already been stated, three sets of corresponding values of tax and gross weight which will indicate the general trend desired in the tax-gross weight relation will serve to fix the constants A, B, and C in the formula.

However, at present there is no generally accepted method for determining the most equitable relation between tax and gross weight or, in this case, forfixing the three sets of values. Their proper evaluation is therefore likely to be the subject of much speculation and debate. They can, of course, be chosen more or less arbitrarily, but the most equitable choice will be assured only by a carefully conducted investigation.

Such an investigation would be concerned first with the determination of the relative costs to the State due to the operation of average vehicles of different gross weights, and second with a consideration of the relative economic and social desirability of the different weight vehicles as units of transportation. Taken together these two factors would fix the justification for taxing corresponding to each gross weight. Taxes could then be made proportional to the justification for taxing.

Gross weight and total tax.—Table 2 outlines one method of determinating quantitatively the equitable average relation between gross weight and tax. The cost to the State due to the operation of motor vehicles D should be estimated by adding the average annual costs over a period of several years of the items E, and F. The item E covers all administrative costs, including registration of vehicles, policing and regulation, signs and traffic devices, and departmental overhead. F is the sum of G and H and includes all highway costs. G represents the costs of constructing, rebuilding, or improving highways, while H represents the costs of patching, repairing, or treating roadways in such manner as to maintain them in proper condition.

The total number of vehicles registered for the average year, I, is divided into three convenient groups according to gross weight, and the number of vehicles in each group (a, b, and c), is recorded.

Administration costs, since they are independent of weight, can be equitably divided equally among all the vehicles in the State, each vehicle being charged with an amount equal to $\frac{E}{I}$.

TABLE 2.—*Outline of a method for determining the equitable total annual taxes for vehicles of different gross weights*

			Total number of vehicles	Total admin. costs	Total constr. costs	Total maint. costs	Total highway costs		Total cost to State	Total revenue required		
			I	E	G	H	F		D	P		
Group	Gross weight, tons		Number of vehicles in group	Administration cost per vehicle	Highway costs			Cost per vehicle	Total cost per vehicle	Tax per vehicle on basis of cost to State alone	Desirability factor	Tax per vehicle modified according to desirability
	Range	Average			Construction	Maintenance	Total					
1	Up to 3	d	a	$\frac{E}{I}$	G_1	H_1	F_1	J	M	Q	T	X
2	3 to 6	e	b	$\frac{E}{I}$	G_2	H_2	F_2	K	N	R	U	Y
3	6 to 15	f	c	$\frac{E}{I}$	G_3	H_3	F_3	L	O	S	V	Z

The difficult and uncertain part of the problem is the distributing of highway costs among the several groups of vehicles in such a manner that each will be charged with that portion for which it is responsible.

Roads designed to carry 3-ton vehicles without undue deterioration would be satisfactory for about 95 per cent of the vehicles in use to-day. To accommodate 4 of the remaining 5 per cent, roads of double this strength would be required, while to be correspondingly safe for the heaviest group, weighing up to 15 tons and constituting about 1 per cent of the total number of vehicles, roads from three to five times as strong would be necessary. In practice, a road of normal design for general traffic will carry, without undue damage, a limited number of the heaviest vehicles now permitted by State laws, and it is only those highways that are used by a considerable number of the heavy vehicles that must be especially designed for their use. However, the additional costs of the stronger roads, where they are built, and any excessive maintenance costs of the lighter roads, where the stronger ones are not provided, are directly chargeable to the relatively small number of heavy vehicles.

In addition, these same heavy vehicles occupy more road space and require considerably wider roads, thus further increasing their share of highway costs.

Of course, costs justified by the size of a vehicle will not be distributed exactly as desired if based on gross weight. Vehicles which are very large but not correspondingly heavy will be undertaxed, while very heavy vehicles of relatively small over-all dimensions will be overtaxed. The few cases of this kind which occur, however, do not warrant the complication of the tax system by including vehicle size as a separate factor.

Distribution of construction and maintenance charges, to be most equitable, should be based on experimental data as to the relative destructiveness of vehicles of different weights on the various types of road. If the average annual mileage covered, the types of road most used, and the amount of traffic encountered are very different for the average vehicles of the different groups, then these factors might also be assumed to vary with weight in the distribution of charges.

In the quantitative apportionment of costs G_3 would be that part of G which could have been saved had there been no vehicles weighing more than 6 tons. In other words, G_3 is the amount expended in building into the roads the necessary extra strength and width to carry the heaviest group of vehicles. In a similar manner an additional amount, G_2, could have been saved had there been no vehicles weighing more than 3 tons, and G_2 is therefore chargeable solely to those vehicles which exceed this weight. G_1, then, is the expenditure for construction which would have provided equivalent improvements had all cars weighed 3 tons or less.

The maintenance costs are divided in like manner. H_1 is the cost of those repairs which would be expected had all vehicles weighed 3 tons or less. H_2 represents the additional damage done by vehicles weighing betwen 3 and 6 tons and H_3 is the amount chargeable solely to the heaviest group of vehicles.

The G and H values should be added to obtain the total highway charges.

F_1, the cost of constructing and maintaining roads for the lightest vehicles, should be divided equally among all vehicles in the State. Therefore $J=\frac{F_1}{I}$. The vehicles in Group 2, together with those in Group 3, should, in addition, be charged with the cost F_2, or $K=J+\frac{F_2}{b+c}$. The vehicles in Group 3 are charged with their share of F_1 and F_2, and in addition F_3 is divided among them solely. Therefore $L=K+\frac{F_3}{c}$.

The total costs to the State per vehicle for vehicles in each of the three groups are M, N, and O and are obtained by adding the administration cost per vehicle, $\frac{E}{I}$, to J, K, and L, respectively.

Special taxes can be made proportional to the total costs per vehicle. As mentioned before the State must fix the total revenue to be collected by means of special motor-vehicle taxes. In Table 2 this total revenue is represented by P. The tax per vehicle should bear the same relation to the cost per vehicle as the total revenue bears to the total cost to the State, or

$$\frac{P}{D}=\frac{Q}{M}=\frac{R}{N}=\frac{S}{O}$$

It is evident from the foregoing discussion that the number of vehicles in a given weight class is an important factor in determining the cost per vehicle. Up to the present time heavy vehicles have been relatively scarce and it is quite possible that taxes on these vehicles, figured on a basis of cost to the State alone, would be prohibitive at the present time. It should be realized, however, that large trucks and busses are built in response to a demand for vehicles which will transport large loads efficiently. A single large truck or bus will carry the load of six or seven small vehicles at a much lower cost per mile and it will occupy less road space and often cause less traffic congestion than the numerous vehicles which it replaces. On account of these advantages, large vehicles are becoming increasingly popular and numerous in the more thickly populated sections. If the number of heavy vehicles continues to increase, if their loads are distributed on six or more wheels equipped with pneumatic tires, and if methods of constructing highways are improved and cheapened, it is quite probable that at some future time the cost per vehicle for this class will be considerably less than at present. For these reasons, prohibitive fees would be highly undesirable.

If it is thought that the cost to the State should not be the only criterion for determining the relation between gross weight and tax, modified taxes (X, Y, and Z) may be obtained by multiplying the values (Q, R, and S) by appropriate "desirability factors" (T, U, and V).

The amounts X, Y, and Z are the taxes which would apply on vehicles whose gross weights are the average gross weights (d, e, and f) of the three groups. The average for each group is obtained by adding the gross weights of all the vehicles in the group and dividing the sum by the number of vehicles in the group.

The three sets of corresponding values of tax and gross weight (X, d), (Y, e), and (Z, f) indicate the trend of the desired equitable relation between gross weight and tax and may be used to evaluate the constants in the formula for total annual tax, as follows:

$$C=\frac{(Z-Y)(e-d)-(Y-X)(f-e)}{(f^2-e^2)(e-d)-(e^2-d^2)(f-e)}$$

$$B=\frac{(Y-X)-C(e^2-d^2)}{e-d}$$

$$A=X-Bd-Cd^2$$

The fact that an effort has been made to tax each vehicle in proportion as that vehicle is responsible for expenses to the State should not be misconstrued as a suggestion that motor vehicles should pay these expenses in full. In practice, the total revenue required from special motor-vehicle taxes (P, Table 2) may be only a small fraction of the total cost to the State (D, Table 2), the difference being paid in general taxes. This is especially true when "desirability factors" differing considerably from unity are thought advisable. In this case the decrease in taxes on the favored classes of vehicles should be compensated for by a low value of P rather than by excessive taxes on the other classes of vehicles.

Fuel tax and registration fees.—The general formula with the numerical values of A, B, and C as determined above represents the total annual tax per vehicle. It would also represent the equitable registration fees for cars traveling the average number of miles per year, if no fuel tax were imposed. Thus, it might logically be used to determine registration fees on electric vehicles which escape the fuel tax. However, in order to distribute taxes fairly, not only according to gross weight but also according to miles traveled, it is most equitable to collect the largest possible portion of the total tax in the form of a fuel tax.

From data on the fuel consumption of a number of different cars it is estimated that fuel taxes distribute themselves among the average gasoline-driven cars of different weights approximately according to the following general formula:

$$\text{Annual fuel tax}=A+B\times(\text{gross weight})$$

where $A=0.0834\times$ (annual mileage in thousands of miles)$\times$(fuel tax in cents per gallon); and $B=0.314\times$ (annual mileage in thousands of miles)$\times$(fuel tax in cents per gallon).

The straight lines showing fuel tax in Figure 2 were obtained from this formula by assuming an average annual mileage of 6,000 miles and various values for the fuel tax in cents per gallon. If, now, the total tax curve (as plotted from the previously determined formula for total tax) is superimposed upon such a fuel tax chart, the most equitable fuel tax is at once apparent. It is the highest fuel tax which can be imposed, without eliminating registration fees entirely on one or more groups of vehicles, and it will be represented by a line just slightly below the total tax curve, as illustrated by the 6 cent line in Figure 2.

The formula for registration fees is now obtained by subtracting the formula for the most equitable fuel tax from the formula for total tax, as explained before.

Registration fees and tire type.—It is fairly well established that solid tires are more destructive to roads than pneumatic tires on a similar vehicle. Just how much more damage they do and just how the ratio changes with the weight of the vehicle or the type of road are not definitely known. However, a somewhat higher tax for solid tires is certainly justified. Since tires have little effect on fuel consumption, the difference must be made in the registration fee.

It is probable that the difference in destructiveness between solid and pneumatic tires is most noticeable in the heavier cars. Therefore a schedule which gives a gradually increasing difference in tax for the heavier vehicles is probably most equitable. Such a schedule can be obtained from the recommended formula by using a higher value of C for solid tired vehicles than for those on pneumatic tires.

Registration fees and number of axles.—When this paper was first written, no recommendations were made in regard to multi-axle vehicles. However, in the past few years vehicles having their weight distributed on three or more load-carrying axles and six or more wheels have increased in number, and since such vehicles are less destructive to roads than four-wheel vehicles of the same gross weight, a reduction in tax for this type appears to be justified. Since the fuel tax does not favor multi-axle vehicles, this reduction must be made in registration fees. A reduction which increases in amount for the heavier vehicles will be most equitable and may be secured, if desired, through the use of a lower value of C in the formula for registration fees.

Obtaining the desired total revenue.—Up to this point the discussion has dealt chiefly with the proper relative distribution of taxes. It may also be necessary to adjust the level of the whole system up or down to produce the required total revenue without changing the relative distribution. The income to be expected from a prospective system should therefore be estimated as closely as possible and the formula for total tax should be multiplied by the proper constant to increase or decrease the total tax per vehicle to the desired amount. From the new formula for total tax thus obtained, the amount of fuel tax and the constants for the registration fee formula must be redetermined.

APPENDIX B

OTHER SYSTEMS CONSIDERED

Early in this investigation the existing methods of determining motor vehicle registration fees were examined. None of these possessed all of the desired requirements. Several other possible methods were devised and rejected before the one described was finally selected as the best available in view of the conflicting requirements of equity, simplicity and adaptability. The limitations of the other systems studied will be discussed in detail. Although the fees and vehicle characteristics used as examples were taken from the tax schedules and vehicle specifications of several years ago they apply, in principle at least, to conditions existing to-day, and are therefore submitted as they were originally prepared.

Gross weight.—States basing registration fees on gross weight generally classify vehicles into groups according to gross weight and impose either arbitrary fees or arbitrary rates per ton or pound for the different groups. An example of an arbitrary schedule of fees is given in Table 3 and shown graphically in Figure 3. Such arbitrary schedules, whether based on gross weight or some other vehicle charactertistic, are likely to show erratic and apparently illogical variations in the rate per ton, and frequently increase by steps which seem excessive or out of proportion to the use of the roads. In the example a vehicle weighing 12,000 pounds pays a fee of $22.50, or about $0.19 per 100 pounds. Another vehicle only a few pounds heavier will have to pay more than three times this fee, or a rate of $0.62½ per 100 pounds, the same as that required for a vehicle of 24,000 pounds gross weight.

TABLE 3.—*Typical arbitrary schedule of fees based on gross weight*

Gross weight in pounds	Fee
5,000 or less	$12.00
Over 5,000 to 12,000	22.50
Over 12,000 to 16,000	75.00
Over 16,000 to 20,000	100.00
Over 20,000	150.00

Table 4 and Figure 4 show a typical system where arbitrary rates rather than arbitrary fees are imposed. This system is an improvement over the arbitrary fee system because rates are more likely to be consistent and differences between successive fees are usually small.

The need for a formula which would be flexible and yet impose gradually changing fees and consistently varying rates automatically regardless of the constants used, led to the selection of the general formula recommended in this report.

Carrying capacity.—If gross weight is accepted as the best single measure of the justification for taxing vehicles—and it is believed that it should be so accepted—then any other characteristic will be a good measure of the justification for taxing only in so far as it is proportional to gross weight. Capacity is only a part of gross weight and in present-day vehicles is only roughly proportional to gross weight.

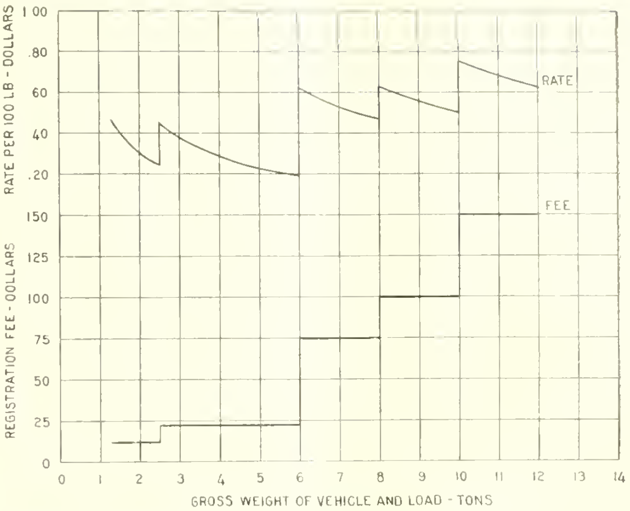

FIGURE 3.—TYPICAL ARBITRARY SCHEDULE OF FEES BASED ON GROSS WEIGHT

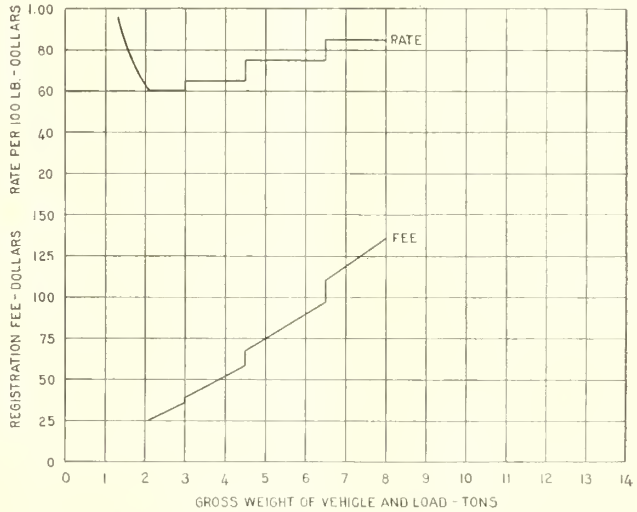

FIGURE 4.—TYPICAL ARBITRARY SCHEDULE OF RATES

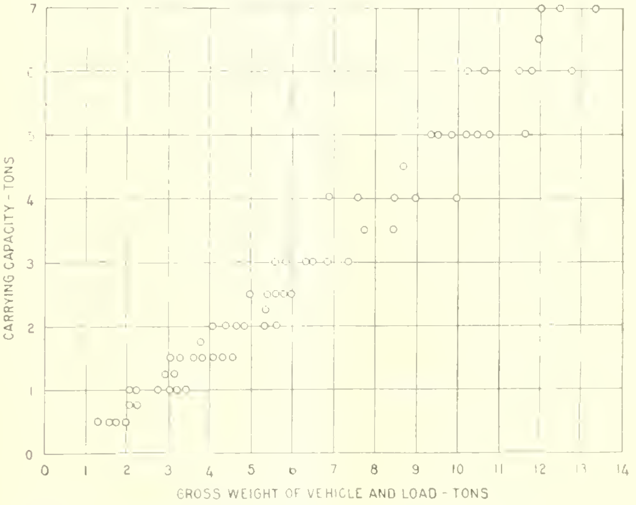

FIGURE 5.—GROSS WEIGHTS AND CARRYING CAPACITIES OF TYPICAL COMMERCIAL VEHICLES

Figure 5 shows graphically how trucks of given capacities vary in gross weight. For example, different makes of 4-ton trucks vary from less than 7 tons to nearly 10 tons in gross weight. In spite of this 6,000 pounds variation in weight, all makes would pay the same fee if rated according to capacity.

It will also be noticed that the heavier trucks of one capacity frequently weigh more than the lighter

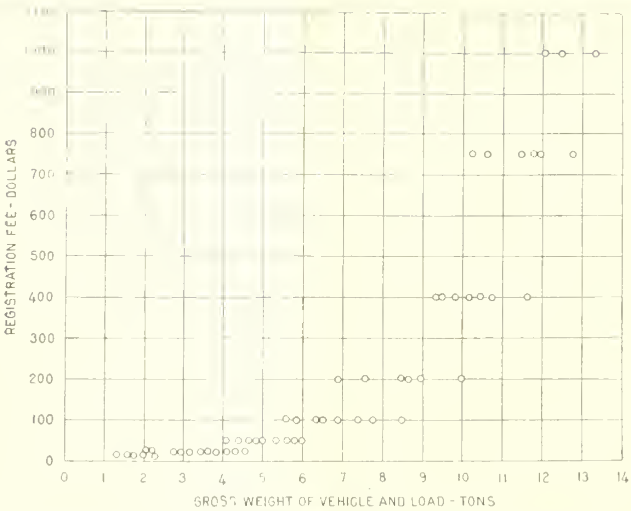

Figure 6.—Example of Commercial Vehicle Fees Based on Carrying Capacity

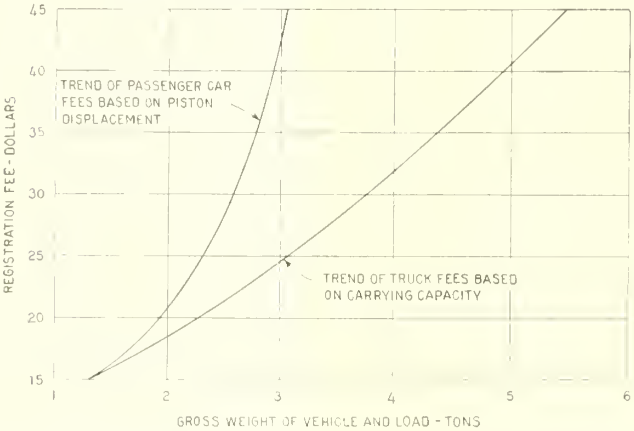

Figure 7.—Example of Use of Different Characteristics for Rating Passenger Cars and Commercial Vehicles

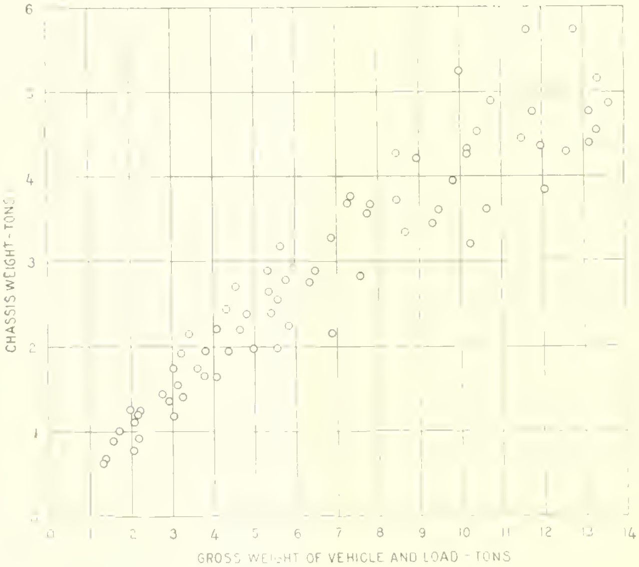

Figure 8.—Gross Weights and Chassis Weights for Typical Commercial Vehicles

Table 4.—*Typical arbitrary schedule of rates*

Gross weight in pounds	Rate in cents per 100 pounds
6,000 or less	60
Over 6,000 to 9,000	65
Over 9,000 to 13,000	75
Over 13,000	85
Minimum fee, $25.	

trucks of the next higher capacity. Thus, the heaviest 5-ton truck listed weighs 23,259 pounds when fully loaded, while the lightest 6-ton truck weighs only 20,450 pounds when fully loaded. Registration will cost this 6-ton truck $350 more in one State and in eight other States at least $100 more than it will cost the 5-ton truck in spite of the fact that the 6-ton truck weighs nearly 3,000 pounds less when loaded and nearly 5,000 pounds less when empty and, therefore, does less damage to the road than the 5-ton truck. Similar inconsistencies occur throughout the whole range of capacities and weights, as shown in Figure 6. A rating which imposes a heavy penalty on 2,000 extra pounds of carrying capacity and neglects entirely variations in vehicle weight of 6,000 pounds is obviously a poor measure of the justification for taxing.

From the viewpoint of economical transportation, it is desirable that vehicles be so designed that their weights empty are a minimum and their carrying capacities a maximum. Basing taxes on capacity tends toward the opposite condition. It discourages large capacities and at the same time fails to penalize overweight vehicles for their inefficient design.

If capacity is used as a basis for taxing trucks, then some other basis must be found for taxing passenger cars. The use of two separate bases is likely to lead to a seemingly illogical discrimination between passenger cars and trucks. Figure 7 illustrates this point graphically. Here passenger-car fees are based on piston displacement and increase rapidly with gross weight. Truck fees are based on capacity and increase much more slowly. The result is that while the lightest passenger cars and trucks pay about the same fee, the heaviest passenger cars pay nearly twice as much fee as trucks of the same weight and the same fee as trucks of nearly twice their weight. In some other States the situation is reversed, and trucks pay the higher fees. For convenience, uniformity, and equity, taxes on all vehicles should be based on the same characteristic.

Chassis weight and unladen weight.—Like the capacity, the chassis weight or the unladen weight of a vehicle constitutes only a part of the gross weight imposed on the roadway. As shown in Figure 8, there is a wide variation in the gross weights of trucks of approximately the same chassis weights. There is a somewhat similar variation in the gross weights of vehicles having approximately the same unladen weights. Therefore, fees will be improperly distributed if based on either of these characteristics.

Figure 9 illustrates the result of basing truck fees on chassis weight. Truck A, with a capacity of 7½ tons, pays the same fee as truck B, with a capacity of only 3 tons, in spite of a difference of nearly 6 tons in their gross weights. Truck C pays twice as much as truck D, although their gross weights are approximately the same, and truck C also pays $75 more than truck A, although truck A is 3 tons heavier when loaded. It is perhaps desirable to tax vehicle weight at a higher rate than pay load, but such excessive penalties as are sometimes involved in the basis of truck fees on chassis weight or unladen weight seem unjustified.

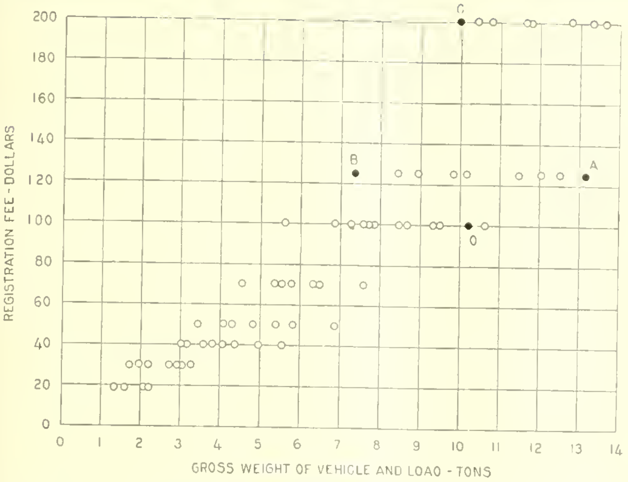

Figure 9.—Example of Commercial Vehicle Fees Based on Chassis Weight

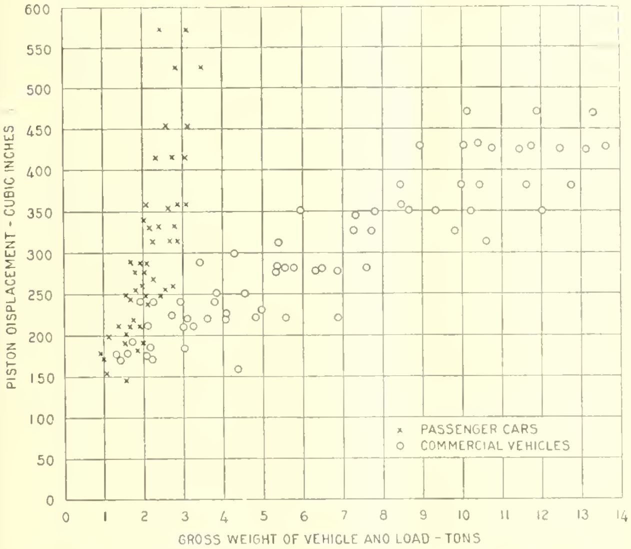

Figure 10.—Gross Weights and Piston Displacements for Typical Vehicles

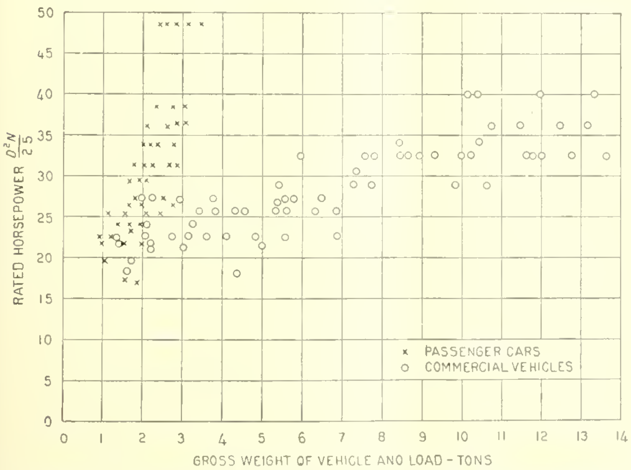

Figure 11.—Gross Weights and Rated Horsepowers of Typical Vehicles

The same criticisms apply to a lesser degree to unladen weight as a basis for taxing passenger cars. Small passenger cars of the same unladen weight have differences in carrying capacity of 3 to 5 persons causing variations in gross weight of about 20 per cent, while the situation for large busses is much the same as for trucks.

Tire width.—When the weight allowed on a wheel or axle is limited by law according to the width of the tires used, tire width tends to become a rough measure of gross weight and may be used as a basis for determining fees.

Tire width seems to have no marked advantages over gross weight as a basis for taxing, but it possesses some disadvantages. If adopted generally, schedules would have to be devised which would distribute taxes equitably among vehicles equipped not only with solid tires but with high-pressure pneumatic and the various classes of balloon tires as well. Such taxes would encourage owners and makers of vehicles to use the minimum width of tire allowed, and those vehicles which used wider pneumatic tires than this minimum would be penalized by the tax in spite of the fact that their equipment was particularly good as regards protection of the highway.

Rated power.—Engine power is available either for moving weight or producing speed, both of which damage the road. Speed, however, for the very great majority of cars is limited not by the power of the engine but by law, by road and traffic conditions, or by the whim of the driver. In States where traffic is relatively dense, it seems probable that the average speed of high-powered cars would differ little from the average speed of low-powered cars, and vehicles which move more slowly than the average speed for all traffic are perhaps as objectionable as those which travel somewhat faster than the average. The value of power as a measure of use of the roads will therefore depend more on its proportionality to gross weight than on its rather doubtful relation to speed.

At present, no power rating available gives values of power which are even approximately proportional to gross weight. Figures 10, 11, and 12 show wide variations in the power ratings of passenger cars of essentially the same gross weight, and great divergence in the weight of trucks of approximately the same power.

An example of the effects of basing passenger-car fees on horsepower is shown in Figure 13. There is a car weighing approximately 4,000 pounds in each of the four fee groups shown. These four cars pay fees of \$12.50, \$20, \$30, and \$40, respectively, supposedly because their engines are capable of propelling them at different speeds. As a matter of fact, their average speeds and their use of the roads are probably nearly the same. Among those cars which pay a \$20 fee is one which weighs only 2,200 and another which weighs 5,500 pounds.

Figures 10, 11, and 12 show how rapidly the power of passenger cars increases with weight and how slowly the power of trucks increases. Obviously the same schedule of fees would not be applicable to both classes. Also, if gasoline cars are to be rated on formula horsepower or piston displacement, different ratings must be used for steam vehicles, for electric vehicles, and for trailers. Taxing each of these classes on a different basis would be almost certain to lead to inconsistencies between the various schedules and discrimination between vehicles in the different classes whose use of the road is approximately the same.

Another difficulty in the use of power as a basis for taxing is that the various power ratings disagree. For

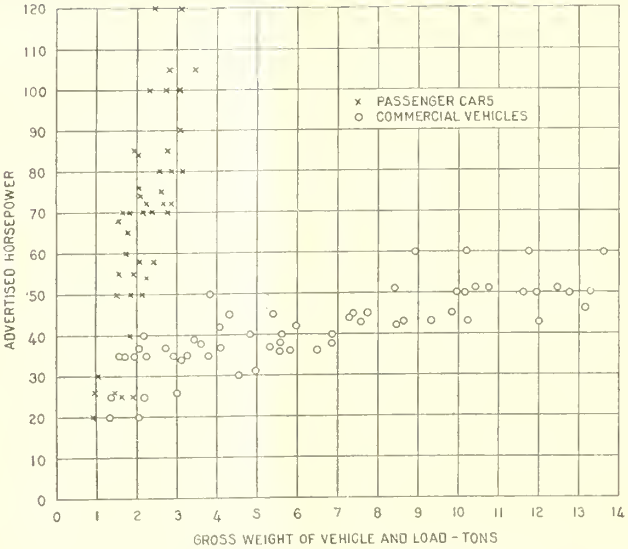

Figure 12.—Gross Weights and Advertised Horsepowers of Typical Vehicles

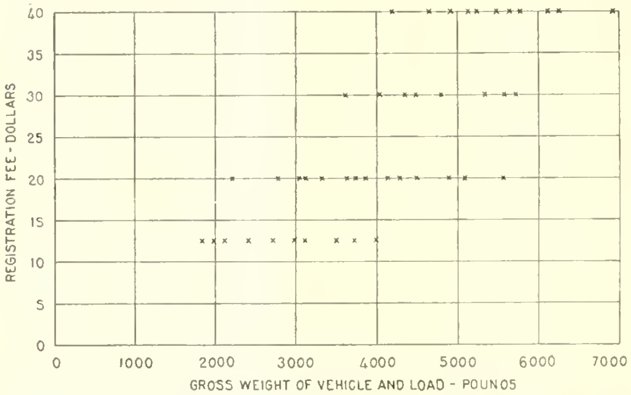

Figure 13.—Example of Passenger Car Fees Based on Rated Horsepower

example, car A has a formula horsepower 17½ per cent greater than that of car B, a piston displacement 47 per cent greater and an advertised horsepower 325 per cent greater. What, then, is the relation on which taxes should be based?

Price and value.—There is, very obviously, no direct relation between the price of a vehicle and its use of the roadways. The only justification for using price as a measure of such use is the remote relation which price happens to bear to gross weight. This relation is shown in Figure 14 and is evidently entirely too remote to provide an equitable basis for taxing. The chart is somewhat similar to those showing the relation between power and gross weight. The average price of passenger cars increases very rapidly with gross weight, while the average price of trucks increases very slowly. A single schedule of fees based on price would therefore not be applicable to both passenger cars and trucks nor would the distribution among units of the same class be equitable because of the wide variation in the prices of passenger cars of approximately the same gross weight and the great divergence in the weights of trucks of approximately the same price.

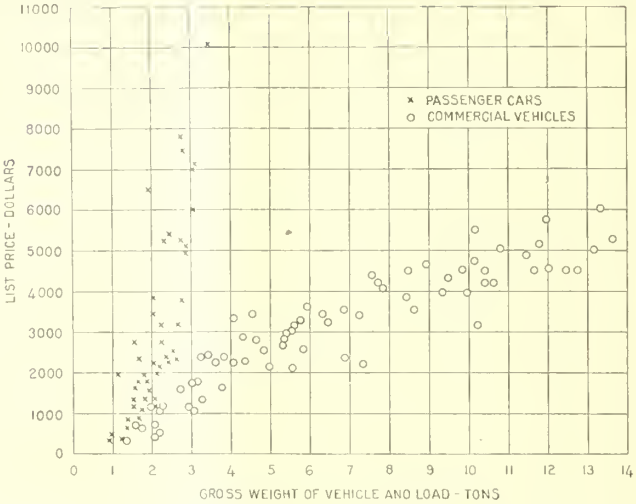

Figure 14.—Gross Weights and List Prices of Typical Vehicles

Value is even more illogical and unsuited as a basis for special taxing. It is odd to conceive of a new truck in perfect mechanical condition and equipped with new tires, being charged with a greater highway expense than an old truck with tires worn down near the legal limit of thickness and mechanical condition a menace to the safety of the public.

Flat rate.—This form of tax is evidently very unfair to the smaller vehicles as it puts as much of a burden on the lightest as on the heaviest cars. Where it is used the gas tax is sometimes expected to correct for the inadequacy of the registration fees on the heavier cars. As explained previously, no fuel tax can do this where an increasing rate or fee per ton is desired.

Combination ratings.—Some States use capacity, unladen weight, power, or value in combination with each other or with gross weight in rating vehicles. The criticisms which apply to the use of these characteristics singly apply also to their use in combination. The use of any of them in connection with gross weight impairs rather than improves the fairness of the gross weight rating and at the same time complicates its administration.

A METHOD OF ANALYSIS OF DATA ON FROST OCCURRENCE FOR USE IN HIGHWAY DESIGN

By J. A. SOURWINE, Senior Highway Engineer, U. S. Bureau of Public Roads

A FEW years ago frost occurrence presented only a minor highway problem. To-day it has become a major problem. Improved facilities for snow removal keep open an increasing highway mileage. Winter traffic is increasing also per unit of mileage. The annually increasing mileage of road left uncovered and unprotected from frost attack makes frost occurrence a highway problem which deserves intensive study.

These facts suggest the question: May frost occurrence and highway frost damage be so interrelated as to make possible the use of existing climatological records as a means for assisting in the determination for a given locality of the probability of occurrence of highway frost damage?

This paper presents a method by which we may so relate and summarize recorded climatic occurrence as to render usable for highway design the records of past weather conditions as a guide in the determination of probable ground freezing occurrence. This study does not consider the problem of freezing weather during the construction period. Nor does it consider the problem of the modifying effect on ground freezing, due to type of soil, to degree of compactness of soil, to varying moisture content in soil, to topographic placement, or to the presence of surface cover which affords continuous shading from the sun. There is considered rather the intensity duration and frequency of low temperature occurrence over a period of years, seeking to develop a method by which climatological records may be made available as a guide in the determination of relative probable highway damage. The determination reached will be relative only, based on the temperature occurrence phase of the ground freezing problem and subject to modification as applied to any local area to meet the actual modifying conditions there existent.

Let us first consider what portion of existing temperature records is most suitable as a basis for comparison to determine the probable relative danger of highway ground freezing for any given location. There seems to be doubt as to the suitability of absolute minimum temperatures[1] for such use. As an illustration, the lowest absolute minimum temperature recorded for Washington, D. C., is $-15°$ F. As a basis for comparison this extreme low minimum temperature does not appear truly typical or as offering a basis suitable for use in highway design for Washington, D. C. The indication is that other modifying factors should also be considered. One of these modifying factors is the relative frequency of low temperature occurrence.

About six years ago the United States Weather Bureau prepared for the Bureau of Public Roads two isothermal charts of the United States, one showing isothermals of normal temperature and the other showing isothermals of average minimum temperature for the month of January. These charts were based on records computed to the end of the year 1914. They represent the existing data available in the files of the United States Bureau of Public Roads on temperature records as affecting highway design in the United States.

A study of summarized climatological data for the United States, taken from Bulletin W of the United States Weather Bureau, shows that January is not always the month of extreme cold, the actual month varying for different localities and even for different years in the same locality, but being in general one of the three months, December, January, or February. In order to establish a common basis for comparison of the relative intensity of low-temperature occurrence in different localities, we will use the records of each observing station for the most severe winter month at that station. The lowest monthly average of daily minimum temperature over the period of record covered by Bulletin W (being in general from the beginning of available record to and including the year 1921) is used as a basis for this comparative study of low-temperature intensities.

CRITICAL INITIAL AIR TEMPERATURE FOR GROUND FREEZING SELECTED AS 23° F.

Before proceeding further we must determine the average critical initial air temperature at which, under conditions of descending air temperature, highway ground freezing occurs. A careful study has been made to determine this critical initial air temperature. Laboratory tests are available, by A. Petit,[2] and by Geo. J. Bouyoucos,[3] in which there was investigated the required lowering of air temperature, in order to produce freezing in soils. The results found by Petit and by Bouyoucos are closely similar. The degree of supercooling of air required varies according to the type of soil, from $-4°$ C. to $-5°$ C. The value found by Petit for an average clay is $-4.8°$ C., or 23.4° F. Both the Petit and Bouyoucos tests show that a duration of supercooled temperature of from 2½ to 4½ hours is required in order to produce freezing in the soil. The period of duration of supercooling as found by Petit is 160 minutes for sand, 190 minutes for average clay, and 266 minutes for peat. For fine, silty clay of the type in which highway ground freezing frequently occurs, three and one-half hours is assumed as an average value of the period of duration of supercooling.

A study of the thermograph records [4] for a number of Weather Bureau stations of the first order located at widely scattered points over the United States shows that an absolute minimum air temperature very seldom endures for a period of three and one-half hours. The maintaining of a given effective low temperature, under conditions similar to a laboratory experiment, for three and a half hours is commonly accompanied, under actual field conditions, by the occurrence of an absolute minimum air temperature, which is 1.5° F. to 2.0° F. lower than the effective temperature required to be maintained throughout the period. Applying this observed relation, together with the results of the laboratory tests by Petit and by Bouyoucos, on the supercooling of air required to produce soil freezing, we determine a critical initial minimum air temperature, for conditions of descending air temperature, at which the average subgrade soil lying in place under nonagitated conditions may be expected to begin to freeze. This critical initial value is from 23.0° F. to 23.5° F. for sand, from 21.4° F. to 22.4° F. for clay, and from 22° F. to 23° F. for fine sandy loams and silty loams. An air temperature of 23° F. is assumed as an average critical initial value marking the beginning of ground freezing in average highway subgrade soil. This value of 23° F. will be used throughout the remainder of this study as the critical initial value of descending air temperature below which the intensities, durations, and frequencies of frost occurrence will be studied and compared.

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[1] Absolute minimum temperature is defined as the lowest temperature occurring during a given cold period.

[2] "Untersuchungen uber den Einfluss des Frostes auf die Temperature—Verhaltnise der Boden," by A. Petit, in "Forschungen auf den Gebiete der Agriculture—Physik," v. 17, p. 285-310.

[3] "Temperature which soils can reach without freezing," by Geo. J. Bouyoucos Journal of Agr. Res., November 15, 1920.

[4] Thermograph records for first order stations, U. S. Weather Bureau.

The next item for determination as a basic relation in our study is the critical depth below the surface of ground for average subgrade soil at which the occurrence of freezing will produce objectionable heaving or will cause unevenness or lack of stability in the highway surface. This critical depth would seem to be properly determined only on the basis of practical field experience. Inquiry was made accordingly. Several highway engineers expressed the opinion that 6 inches of ground freezing is a critical value and that at this depth ground freezing becomes objectionable, basing such opinions on the tendency of present-day design of pavement depths for first class pavements. Other highway engineers expressed a judgment favoring a 4-inch depth as critical, basing the conclusion on experience with lesser thicknesses of pavement and on conditions in surface-treated roads. For the purpose of this study a critical depth of 3 inches has been assumed as the depth below which ground freezing becomes a problem for consideration in highway design.

CRITICAL LOWEST MONTHLY MINIMUM TEMPERATURE FOR OBJECTIONABLE GROUND FREEZING DISCUSSED

In order to use the records of lowest monthly average of daily minimum temperatures as a basis for comparative study to determine the relative danger of highway ground freezing for any locality, we must determine the critical value of lowest monthly average of daily minimum temperature at which ground freezing occurrence passes the allowable frequency for 3-inch depth. As to what constitutes allowable frequency, we are faced with the necessity for an arbitrary assumption. For this study we assume an allowable frequency of 5 per cent. We define "5 per cent allowable frequency" to mean that when we consider for any locality all cold periods sufficient to cause freezing of average surface soil, ground freezing below 3-inch depth may occur 1 time in 20. A frequency of more than 5 per cent, or more than 1 period in 20, we designate as "objectionable frequency."

Let us now determine a tentative critical value of the lowest monthly average of daily minimum temperatures which corresponds with an average of 5 per cent frequency of occurrence of highway ground freezing to a depth greater than 3 inches. Reference is again made to the tests of Petit and of Bouyoucos, consideration being given this time, not to the supercooling of air, but to the supercooling of soil required before soil freezing will occur. The amount of supercooling of a nonagitated soil required for freezing is found to vary from −1.5° C. for sand to −2.0° C. for clay, with a required duration of three to four hours. This is equivalent to the requirement which is assumed as applying to soils ranging from fine silty clay loam to clay, that an effective soil temperature of 28.4° F. shall be constantly maintained over a period of three and one-half hours in order to produce objectionable soil freezing. In the study of United States Weather Bureau thermograph records of air temperature occurrence it has been found that for cold periods of three and one-half hours below a given effective temperature the absolute minimum temperature was on the average 2° F. below the maintained effective temperature. Assuming a similar relation to apply to soil temperatures as found for air temperatures, a minus correction of 2° F. is applied to the effective soil temperature of 28.4° F. above found, which gives a required absolute minimum soil temperature of approximately 26.4° F., coincident with the inception of ground freezing for the general type of subgrade soil in which troublesome ground freezing frequently occurs.

RELATION ESTABLISHED BETWEEN MINIMUM TEMPERATURE IN AIR AND THE SOIL

The next step is to establish a relation between average minimum air temperature and average minimum temperature in soil at a 3-inch depth. For the most complete investigation of this subject reference is made once more to Bouyoucos, and particularly to a series of field measurements conducted by him at East Lansing, Mich., during which he measured temperatures in air 4 feet above the surface and in ground 3 inches below the surface[5] throughout the month of

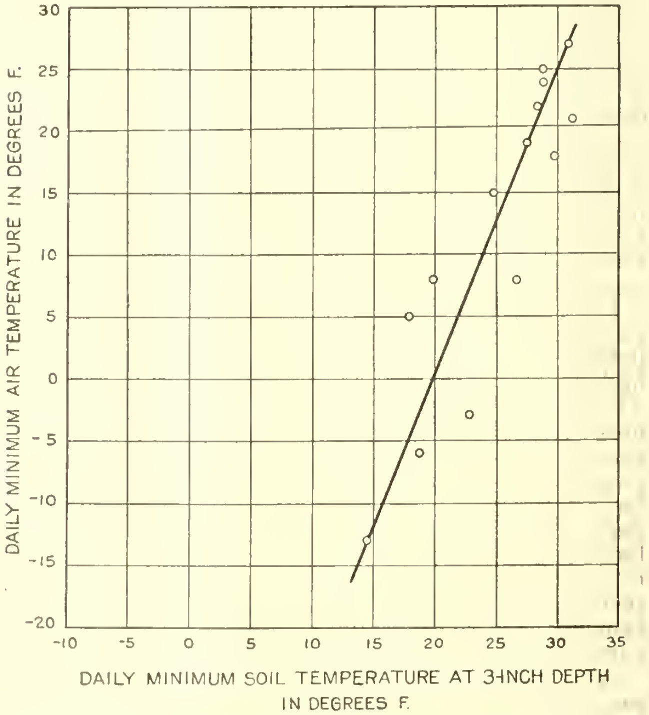

Figure 1.—Daily Minimum Soil Temperature at 3-Inch Depth Compared With Daily Minimum Air Temperature, During a Period of Descending Temperature

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[5] "Soil Temperatures," by Geo. J. Bouyoucos Bulletin No. 26 Mich. Agr. Coll Exp. Station.

January, 1915. Daily minimum, maximum, and mean temperatures were observed and recorded during this test. For 16 days of the month the record shows a descending temperature. Figure 1 shows the minimum temperature in air compared with that in the soil at a 3-inch depth for each day of descending temperature during the month.

Through the points of actual occurrence, plotted in Figure 1, a line was drawn representing the average relation between minimum air temperature and minimum temperature in soil at a 3-inch depth. From this average line the average minimum air temperature, which is equivalent to a soil temperature of 26.4° F. at a 3-inch depth, is found to be 16° F.

FREQUENCY OF MINIMUM TEMPERATURES STUDIED

The above derived value of 16° F. is the actual average equivalent minimum air temperature at which ground freezing begins at 3-inch depth. At this minimum air temperature of 16° F. no allowable frequency of frost occurrence is permitted below 3-inch depth. We desire to use for design an allowable frequency of 5 per cent for frost occurrence below 3-inch depth. Let us study the existing relations and determine the absolute minimum air temperature at which 5 per cent of frost occurrence below 3-inch depth is permitted. For this study we select 15 first-order stations of the United States Weather Bureau, rather well distributed across the United States, so as to obtain typical data representative of the different sections of the country.

The stations selected are all located in regions classified as being in doubtful danger from ground freezing and are well distributed over the United States so as to obtain data representative of the different sections of the country. From the records of these 15 stations there were tabulated all periods where the temperature fell below 23° F. (critical initial air temperature assumed as required to produce freezing in average subgrade soil) for a duration of eight hours or more, and for a drop in temperature greater than 3° F. The tabulation showed the minimum temperature in degrees Fahrenheit for each cold period and the duration of cold period below 23° F. For four of the stations the record covered the 15-year period from 1907 to 1921 (for Tonopah, Nev., 14 years only, on account of no record for 1907 available). For the other 11 stations the record covered an 11-year period, from 1911 to 1921. These records were summarized to show frequency of cold period and low temperature occurrence, and a minimum temperature-frequency curve was plotted for each station. Typical curves showing the variation in frequency for varying minimum temperatures are shown in Figure 2. The ordinate of any point in Figure 2 denotes the relative frequency of occurrence of the absolute minimum temperature indicated by the abscissa. This relative frequency is expressed as a percentage of the total number of cold periods. As an illustration, in the case of Washington, D. C., the point on the curve for that station having an abscissa of 0° has an ordinate of 3 per cent, indicating that, of all cold periods occurring at Washington, D. C., for only 3 per cent of the number did the temperature go as low as 0° F. From the curve for each station a minimum temperature value is obtained corresponding to 5 per cent frequency, and from the available records the absolute minimum temperature reported during the period covered by the study is determined. Table 1 shows these two values and the average difference between them for each of the 15 typical stations.

TABLE 1.—*Minimum temperature of 5 per cent frequency occurrence versus absolute minimum temperature*

Station	Period of study	Minimum temperature with frequency of 5 per cent (1)	Absolute minimum temperature (2)	Difference (1)−(2)
		° F.	*° F.*	*° F.*
New Haven, Conn.	11 years, 1911–1921	−0.5	−12.6	−11.5
New York, N. Y.	do	+1.0	−14.0	−15.0
Philadelphia, Pa.	do	+4.0	−4.0	−8.0
Washington, D. C.	15 years, 1907–1921	+5.0	−13.0	−18.0
Elkins, W. Va.	11 years, 1911–1921	−5.0	−28.0	−23.0
Lynchburg, Va.	do	+4.0	−7.0	−11.0
Lexington, Ky.	15 years, 1907–1921	−2.0	−14.0	−12.0
Nashville, Tenn.	11 years, 1911–1921	+1.3	−10.0	−11.3
Evansville, Ind.	do	−1.8	−16.0	−14.2
Cairo, Ill.	do	−2.0	−15.0	−13.0
St. Louis, Mo.	do	−3.5	−16.0	−12.5
Bentonville, Ark.	do	−2.5	−20.0	−17.5
Oklahoma City, Okla.	do	−0.5	−10.0	−9.5
Amarillo, Tex.	15 years, 1907–1921	+1.0	−11.0	−12.0
Tonopah, Nev.	14 years, 1908–1921	+5.0	−7.0	−12.0
Average				−13.37

CRITICAL VALUE OF LOWEST MONTHLY AVERAGE OF DAILY MINIMUM TEMPERATURE FOUND TO BE 23° F.

Since it was observed that one station (Elkins, W. Va.) in the table shows a difference considerably above the average and another station (Philadelphia) shows a difference somewhat below the average, an average value of difference was computed, excluding values for Elkins and Philadelphia, and was found to be 13.04° F. As a representative difference between absolute minimum temperature and minimum temperature of 5 per cent frequency occurrence, we will assume the value 13° F.

The air temperature at which ground freezing begins at 3-inch depth in average subgrade soil has been found to be 16° F. By applying to this value the average difference of 13° F. between absolute minimum temperature and minimum temperature for 5 per cent frost frequency, 3° F. is obtained as a critical absolute minimum air temperature coincident with 5 per cent frequency of ground freezing at a 3-inch depth.

By the use of data from the thermograph records of the four stations of which a 15-year study was made, Figures 3 to 5 were prepared showing the relation between the monthly average of daily minimum temperature and absolute minimum temperature for each station. A line was drawn in each instance representing the average location of these points. From this line there was obtained for each station the monthly average of daily minimum temperatures corresponding to an absolute minimum of 3° F. The results are as follows:

	° F.
Washington, D. C.	24.2
Amarillo, Tex.	24.4
Tonopah, Nev.	23.3
Lexington, Ky. (graph not shown)	25.0
Average	24.4

The above is the monthly average of daily minimum temperature for the period of the thermograph study (15-year period 1907–1921). A tabulated comparison of lowest monthly average of daily minimum temperatures for the 15-year period compared with the lowest monthly average of daily minimum temperatures over the full period of United States Weather Bureau records is shown in Table 2.

Table 2 indicates that we should apply a correction of −2° F. to the lowest monthly average of daily minimum temperature for the 15-year period in order to obtain the lowest monthly average of daily temperature over the period of United States Weather Bureau

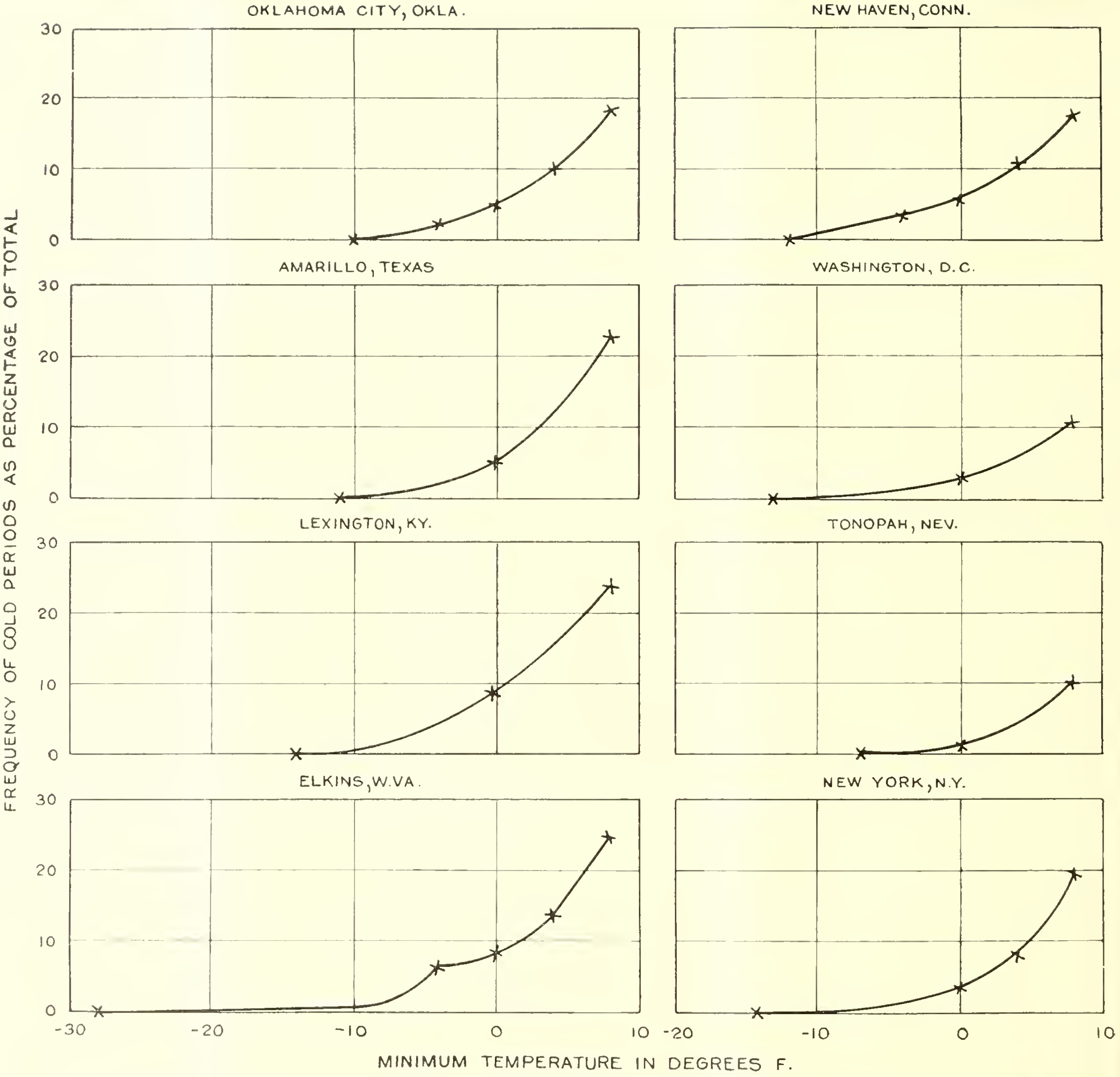


FIGURE 2.—MINIMUM TEMPERATURE-FREQUENCY CURVES FOR SELECTED STATIONS SHOWING RELATION BETWEEN MINIMUM TEMPERATURE OCCURRING WITH A FREQUENCY OF 5 PER CENT, AND ABSOLUTE MINIMUM REPORTED

TABLE 2.—*Relation between lowest monthly average of daily minimum temperature for 15-year period 1907–1921 compared with lowest monthly average of daily minimum temperature for total period of United States Weather Bureau records*

Station	Average for 15-year period	Average for total period of U. S. Weather Bureau records
	° F.	° F.
Washington D. C	25.4	25.7
Amarillo Tex	26.0	20.7
Tonopah Nev. [1]	24.0	24.1
Lexington Ky	26.2	23.2
Average	25.4	23.4

[1] 14-year record 1908–1921.

record. Since the study was based on a small number of stations and there was some difference in the variation shown by the different stations, approximately two-thirds the amount of the indicated correction will be applied, making a correction of − 1.4° F. With this correction applied, the critical value for monthly average of daily minimum temperature over the entire period of United States Weather Bureau record is determined as 23° F. This critical value of 23° F., for the lowest monthly average of daily minimum temperature, is assumed as a critical design value for highway ground freezing.

In the compilation of the temperature data shown in Figure 6 use was made of data from all stations for which the United States Weather Bureau has summarized temperature records prior to and including the year 1921.

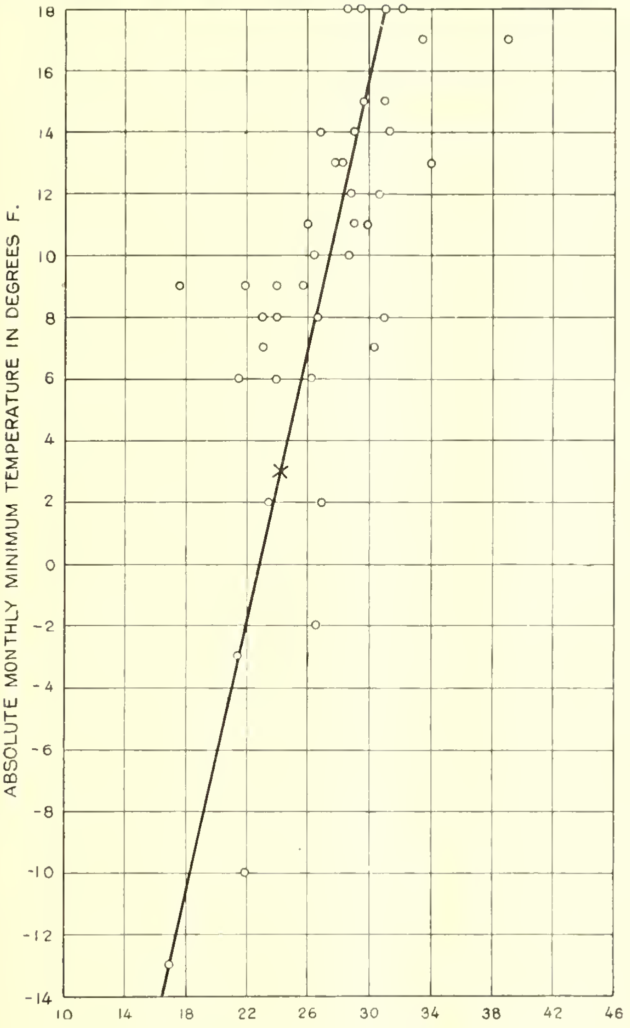


FIGURE 3.—RELATION BETWEEN MONTHLY AVERAGE OF DAILY MINIMUM TEMPERATURE AND ABSOLUTE MINIMUM TEMPERATURE AT WASHINGTON, D. C.

(Bulletin W, United States Weather Bureau.) An isothermal map was made for each State, and these maps were combined in Figure 6. On this map the isohyetal lines or lines of low average winter rainfall, indicating relatively arid regions, are taken from the Atlas of American Agriculture, United States Department of Agriculture. The rainfall shown is the average winter rainfall which occurred during three winter months, December, January, and February. All low average winter rainfall of 2 inches and less is shown. The areas with an average winter rainfall of 2 inches or less and within a zone having a lowest monthly average of daily minimum temperature between 10° F. and 23° F. are cross-hatched on Figure 6 as being areas which, because of their winter aridity and of their relatively moderate range of extreme cold, may be areas of doubtful danger from the standpoint of highway ground freezing. The presentation of these isohyetal lines marking arid regions with low winter rainfall is made to invite attention to this phase of climatic occurrence, with the suggestion that further more detailed study of these regions is advisable. The assumed temperature range of 10° F. to 23° F. is approximate only and subject to future modification as to proper lower limits of temperature range.

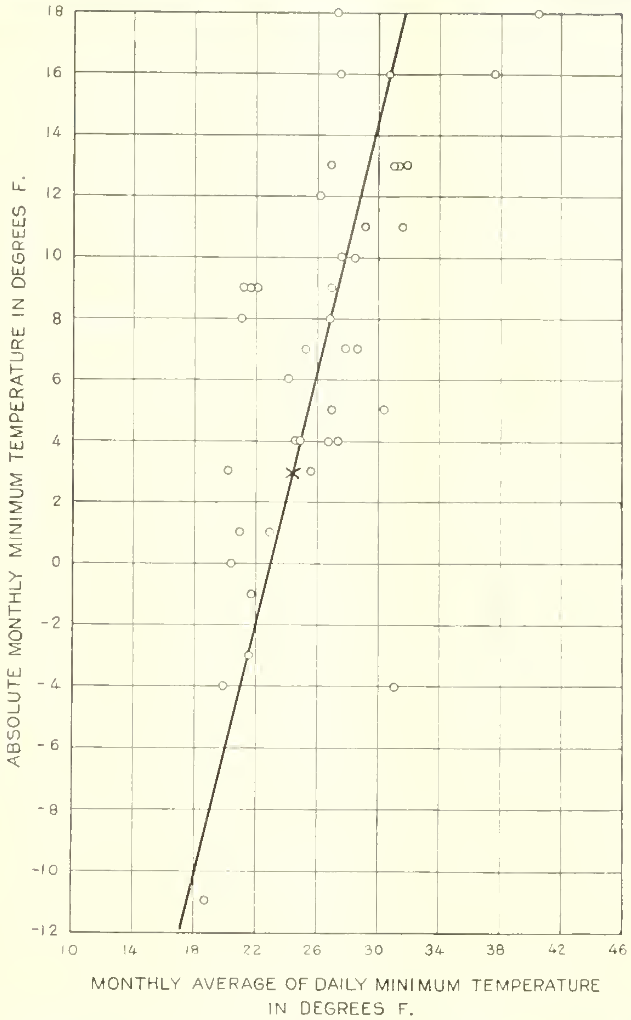


FIGURE 4.—RELATION BETWEEN MONTHLY AVERAGE OF DAILY MINIMUM TEMPERATURE AND ABSOLUTE MINIMUM TEMPERATURE AT AMARILLO, TEX.

Summarizing the preceding phase of the study, in which only low-temperature intensities and frequencies are considered and a 5° allowable frequency of ground freezing below 3-inch depth is assumed, the 23° F. isothermal shown in Figure 6 marks a general critical temperature line below which should lie areas relatively safe and above which should lie areas relatively dangerous from highway ground freezing. The critical temperature line thus shown and marked is a general line only based on the recorded intensity and frequency of low temperature occurrence applied to average highway conditions and subject to modification or approval for any local area as shown by detail study covering local conditions of soil, drainage, surface cover, and exposure.

DURATION OF COLD PERIODS AN IMPORTANT FACTOR

A further study has been made taking into account the duration phase of cold period occurrence. The effect of duration of cold period in accentuating and rendering more severe the damage from cold is a matter of common knowledge. It is well known that several days duration of low temperature, even with a rather moderate minimum temperature, may produce effects equally damaging and possibly even more damaging than those produced by a minimum temperature several degrees colder but having only a short duration. The problem has been to relate and express the different

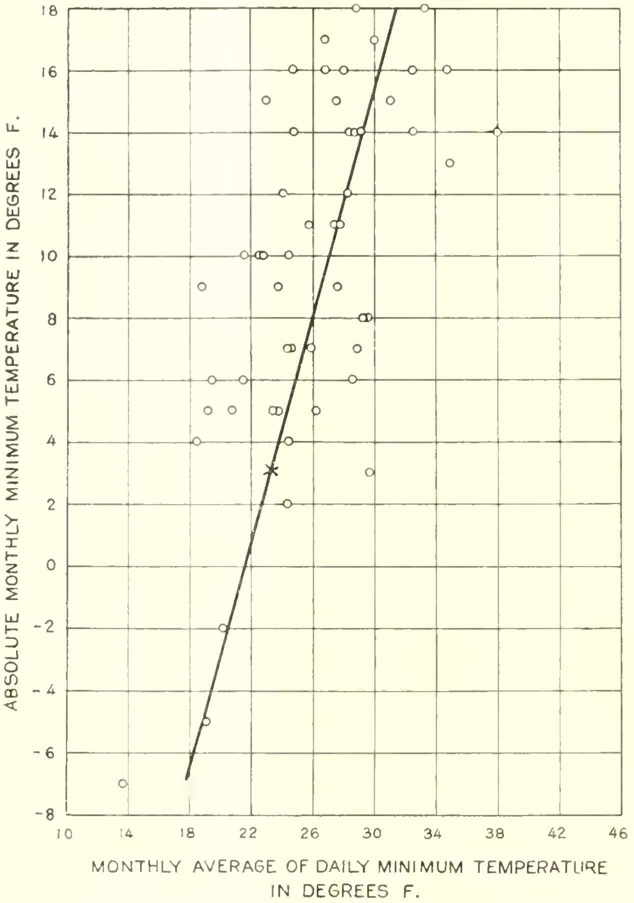


Figure 5.—Relation Between Monthly Average of Daily Minimum Temperature and Absolute Minimum Temperature at Tonopah, Nev.

variables of low-temperature intensity, frequency, and duration in such a way as to make possible a means of measuring the relative degree of their combined effect.

For the purpose of this study a method of measuring the combined effect of low temperature and of cold period duration has been worked out and applied as a means of comparative measurement of cold period occurrence for different periods and in different localities. The unit used for comparison is similar to the "ton-mile" so common in highway traffic terminology. The term used is "degree-hours." The degrees are measured in °F. below the critical initial air temperature and the duration is measured in hours of cold period below the critical initial air temperature. The approximate value of "degree-hours" for any given cold period is $\frac{1}{2} D T$, where D equals duration in hours below critical initial air temperature and T equals the degrees Fahrenheit of absolute minimum air temperature below critical initial air temperature. For this study the critical initial air temperature used is 23° F.

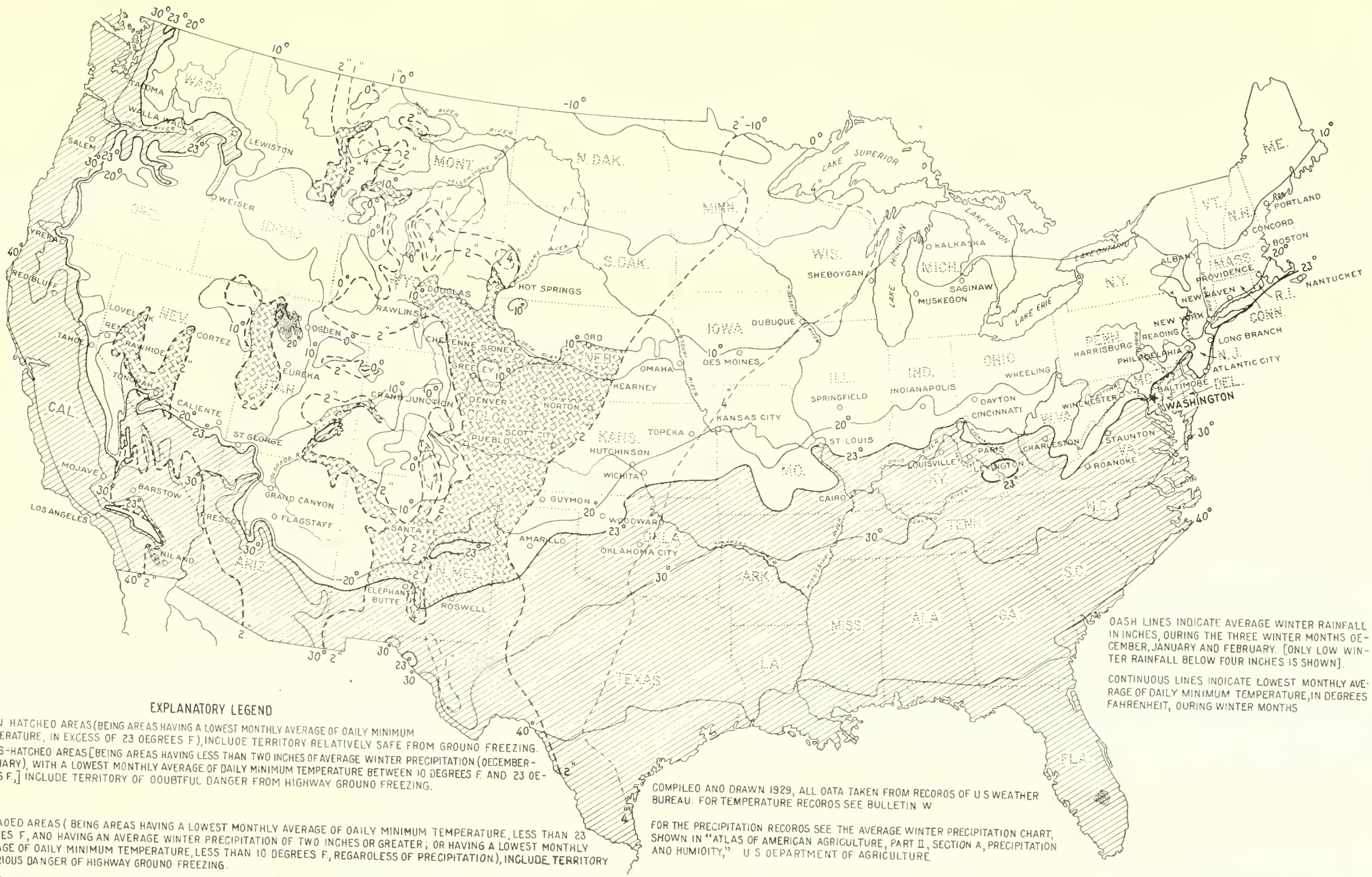
A "degree-hour" study of cold periods has been made, using data from 35 Weather Bureau stations lying in the general proximity of the tentative critical isothermal (23° F.) shown in Figure 6. The thermograph records from each of these stations, covering a period varying for different stations from 5 to 15 years, were studied. The first study covered a 5-year period, but this length of time was in general found to be insufficient to give a representative record. A study covering a 15-year period was then made for a few stations, and an 11-year period was studied for other stations located in critical positions, in determining the position of equal degree-hour lines. Five-year records from a few minor stations were also studied. All data were summarized according to frequency of occurrence both of relative duration of cold period and of relative minimum temperature of cold period. There was then plotted for each station a duration-frequency curve and a minimum temperature-frequency curve. From the two frequency curves thus plotted there was derived a third curve, the degree-hour-frequency curve (fig. 7).

As in Figure 2, the ordinate of any point in Figure 7 denotes relative frequency of occurrence expressed as a percentage of the total number of cold periods for the given station. For the minimum-temperature frequency curve the abscissa denotes absolute minimum temperature of cold period, as in the case of Figure 2. For the duration-frequency curve the abscissa denotes duration of cold period. For the degree-hour frequency curve the abscissa denotes degree-hours accumulated during cold period.

A measure of the quantity of cold occurring at each station was determined by selecting as an index the degree-hours corresponding to a frequency of 5 per cent, the numerical value being taken from the degree-hour-frequency curve. These degree-hour indices for the different stations were plotted on a map of the United States (fig. 8), which for convenience of identification has been called an "isothermo-chronal map." On this map, isothermo-chronal lines, or lines of equal "degree-hour-index" for 5 per cent frequency of occurrence, have been drawn. After considerable study, a degree-hour index of 900 was assumed as the critical isothermo-chronal, representing the danger line for highway ground freezing, based on the combined study of low temperature intensity, cold period duration, and cold period frequency. For convenient presentation of the variation in the "degree-hour" index with change in location, one isothermo-chronal on each side of the critical isothermo-chronal has been shown, the isothermo-chronals of 600 and of 1,200 degree-hours being used. It will be noted that this degree-hour index is very sensitive, changing greatly for relatively small differences of location, or of cold period occurrence.

INDEX MAP USED TO SHOW MOST ADVERSE CONDITIONS OF FROST OCCURRENCE, CONSIDERING THE THREE PHASES OF INTENSITY, FREQUENCY, AND DURATION

A comparison of the isothermal study as shown in Figure 6 and the isothermo-chronal study as shown in Figure 8 shows a considerable area in the eastern and central portions of the United States not indicated as dangerous for ground freezing by the 23° isothermal, but which is definitely indicated as in danger of ground freezing, when considered from the combined standpoint of low temperature and of period of duration of cold. The Columbia River Valley in Washington and Oregon appears to offer a somewhat similar condition, but the limited number of first-order Weather Bureau stations located in this region do not afford a basis for other than approximate location of isothermo-chronal lines. In the mountainous areas in northwestern Texas, New Mexico, Arizona, Nevada, eastern California, and western Oregon and Washington the number of first-order weather stations is relatively small. The data obtained from the few stations available show that most of the cold periods are of relatively short duration, and therefore the period of duration is not in general an important factor. In this region the critical isothermal is in general the line of demarcation between relatively dangerous territory and territory relatively safe, and isothermo-chronal lines are not shown.



EXPLANATORY LEGEND

1 PLAIN HATCHED AREAS (BEING AREAS HAVING A LOWEST MONTHLY AVERAGE OF DAILY MINIMUM TEMPERATURE, IN EXCESS OF 23 DEGREES F), INCLUDE TERRITORY RELATIVELY SAFE FROM GROUND FREEZING.

2 CROSS-HATCHED AREAS [BEING AREAS HAVING LESS THAN TWO INCHES OF AVERAGE WINTER PRECIPITATION (DECEMBER-FEBRUARY), WITH A LOWEST MONTHLY AVERAGE OF DAILY MINIMUM TEMPERATURE BETWEEN 10 DEGREES F. AND 23 DEGREES F.,] INCLUDE TERRITORY OF DOUBTFUL DANGER FROM HIGHWAY GROUND FREEZING.

3 UNSHADED AREAS (BEING AREAS HAVING A LOWEST MONTHLY AVERAGE OF DAILY MINIMUM TEMPERATURE, LESS THAN 23 DEGREES F, AND HAVING AN AVERAGE WINTER PRECIPITATION OF TWO INCHES OR GREATER; OR HAVING A LOWEST MONTHLY AVERAGE OF DAILY MINIMUM TEMPERATURE, LESS THAN 10 DEGREES F, REGARDLESS OF PRECIPITATION), INCLUDE TERRITORY IN SERIOUS DANGER OF HIGHWAY GROUND FREEZING.

DASH LINES INDICATE AVERAGE WINTER RAINFALL IN INCHES, DURING THE THREE WINTER MONTHS DECEMBER, JANUARY AND FEBRUARY. [ONLY LOW WINTER RAINFALL BELOW FOUR INCHES IS SHOWN].

CONTINUOUS LINES INDICATE LOWEST MONTHLY AVERAGE OF DAILY MINIMUM TEMPERATURE, IN DEGREES FAHRENHEIT, DURING WINTER MONTHS

COMPILED AND DRAWN 1929, ALL DATA TAKEN FROM RECORDS OF U S WEATHER BUREAU. FOR TEMPERATURE RECORDS SEE BULLETIN W

FOR THE PRECIPITATION RECORDS SEE THE AVERAGE WINTER PRECIPITATION CHART, SHOWN IN "ATLAS OF AMERICAN AGRICULTURE, PART II, SECTION A, PRECIPITATION AND HUMIDITY," U S DEPARTMENT OF AGRICULTURE

FIGURE 6.—ISOTHERMAL GUIDE MAP FOR USE IN DETERMINING AREAS SUBJECT TO GROUND FREEZING. COMPILED FROM U. S. WEATHER BUREAU RECORDS

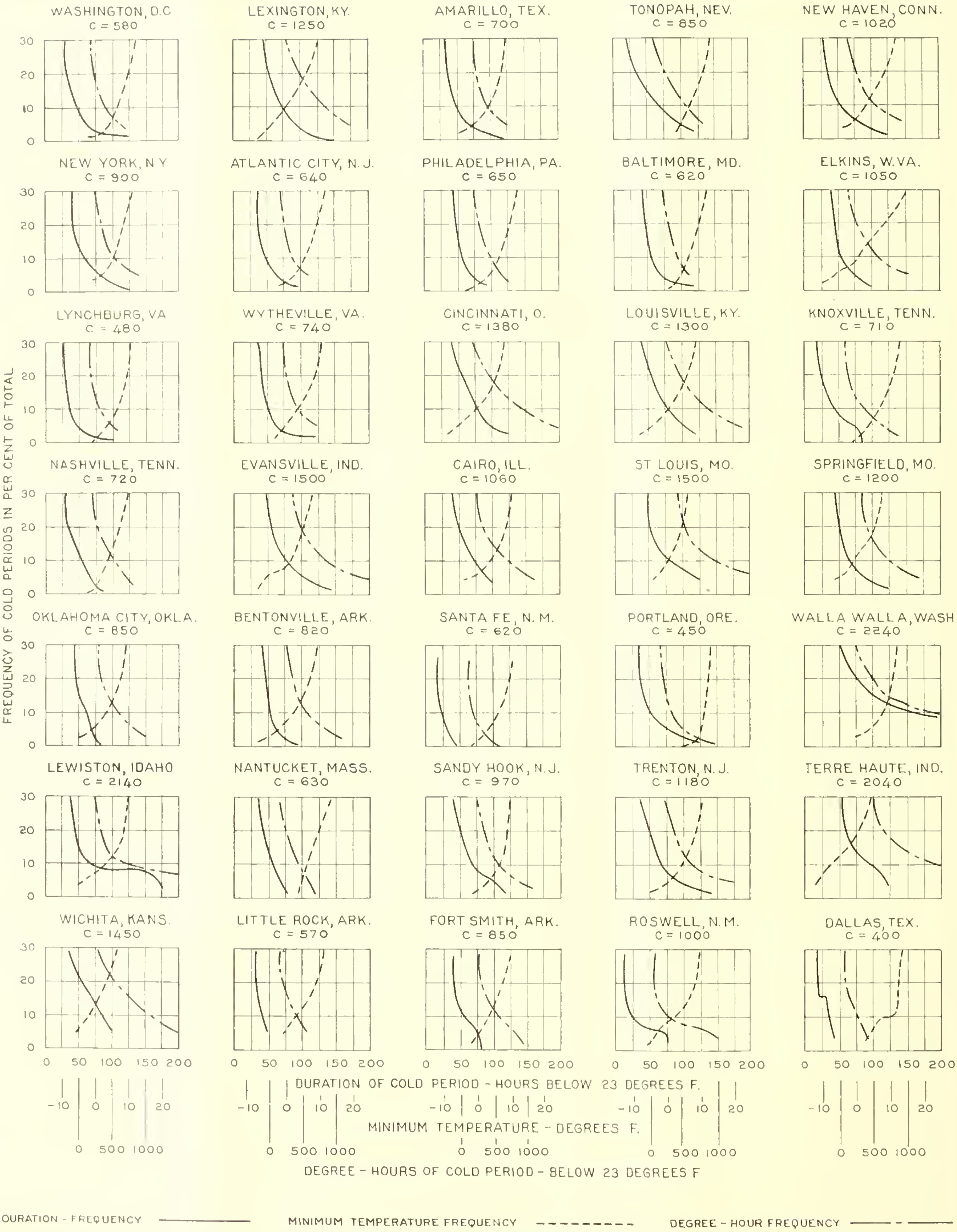


Figure 7.—Duration-Frequency Curves, Minimum Temperature-Frequency Curves, and Degree-Hour-Frequency Curves for 35 Selected Weather Bureau Stations

The general problem of temperature study as affecting highway design has been, for convenience of consideration, divided into two subdivisions of study, the one completed study being shown in graphic presentation by the use of a series of isothermal lines and the other by a series of isothermo-chronal lines. From the facts developed in the course of the work on the two subdivisions of study it now seems advisable to take up a comparative study of the two and consider the results as two parts of one general problem. As a means for so doing, it was found convenient to prepare a third map (fig. 9) showing an index line marking probable objectionable frost occurrence across the United States based on the most adverse conditions of occurrence as indicated by a combined study of the isothermals (shown in fig. 6) and of the isothermo-chronals (shown in fig. 8). For convenience of reference Figure 9 has been designated as a "highway ground-freezing index map" of the United States. It will be noted that no isohyetals are shown in Figure 9, as these data have already been presented in Figure 6.

PROPOSED EXTENSION OF METHOD USED IN THIS PAPER

The question has arisen as to whether a similar method of analysis and digest may not be used to show the relative depth of ground-freezing occurrence for various localities throughout the United States. A research of this sort, following the method used in this paper and accompanied by tabular presentations and charts, might serve as a basis for the study of subgrade soils from the viewpoint of frost occurrence. As in the present case, such a study would be based on an assumed standard average condition of soil type, soil moisture content, degree of compactness of soil, conditions of cover and shade, degree of exposure to sun and to wind, topographic placement as to altitude and slope, and other modifying conditions. This proposition is not treated in the present paper but has arisen from a discussion thereof, and it is suggested as a matter which might well be given study in the future.

CONCLUSIONS SUMMARIZED AND APPLICATIONS DISCUSSED

A brief summarized discussion of the above presented matter and of another related study may be pertinent. In the development of this study for presentation as a method available for practical use by the highway engineer dealing with the frost-occurrence phase of the ground freezing problem, the endeavor has been throughout to present a digest of frost-occurrence facts. The primary objective has been to develop a method by which the records of cold period occurrence in the United States may be presented in simple and readily usable graphic form. The few simple charts developed are intended for use as a reference guide through the use of which the frost-occurrence phase of the highway ground-freezing problem may be readily related to, and for any given location of local area studied jointly with, the other modifying phases of the entire general problem of highway ground-freezing occurrence. A study of other modifying influences which affect ground freezing, including variation in soil types, soil moisture content, conditions of cover or exposure, conditions of geographic or topographic placement, is not included as part of the objective of the present paper. It has been sought merely to present the facts of cold-period occurrence as obtained from a digest of existing weather records, and to work out a method of presentation of the relation of cold period occurrence to highway ground freezing occurrence by which the existing records of climatological data may be rendered conveniently applicable for study of the ground-freezing problem as affecting highway design. The three phases of presentation herein made, comprising as a whole a method for applying records of temperature occurrence to the ground freezing problem of highway design, are as follows:

(1) An isothermal study, considering the problem from the standpoint of the intensity and frequency of low temperature occurrence.

(2) An isothermo-chronal study, considering the problem from the combined standpoint of low temperature intensity, cold-period duration, and frequency of occurrence

(3) A highway ground-freezing index study, combining the most adverse conditions affecting highway ground freezing as developed by studies (1) and (2) and presenting the result graphically in the form of a highway-ground-freezing index line for the United States, based on the existing records of past weather occurrence.

It should be noted that the general analysis of this problem and the method above outlined are not dependent upon any values assumed or computed. For the few values assumed as part of the development of the study the actual range of possible variation is relatively small. Minor modification of one value or another would not alter largely the final position of the tentative highway ground-freezing index line. The selection of a critical depth of 4 inches instead of 3 inches, below the surface, if other assumed values were unchanged, would produce an index line for objectionable highway ground freezing approximately parallel to and located a few miles north of the index line as developed in the present study. The selection of a different critical depth, however, would bring up at once the question whether the allowable frequency ought not also to be modified, a less value of allowable frequency being assumed for frost occurrence passing the 4-inch depth than has been allowed for frost occurrence passing the 3-inch depth. If it were decided that such a modification should be made, a compensating factor for the change in critical depth would be thus introduced and the resulting highway ground-freezing index line would remain relatively unchanged from the position of highway ground-freezing index line, as developed in the present study.

It will be noted that the method as outlined is equally adaptable to the values assumed and computed or to such modified values as may be assumed or computed. It will be noted also that the presentation of the facts of cold occurrence is in such form as to be available for use in any given locality with such minor modification of critical values as will fit local conditions, according to the experience and judgment of the local highway authorities. The establishment of a more or less severe standard of allowable frequency of frost occurrence beyond a given depth based on an economic study, comparing probable highway ground-freezing damage as against estimated additional construction cost required to prevent the occurrence of that damage, becomes possible by variation of the critical value of isothermal or isothermo-chronal for any given locality. With the critical value altered the study may proceed, using for the frost-occurrence phase of the study the same charts and maps which would have been used had the tentative critical values of isothermal and isothermo-chronal developed in the course of the present study been found suitable. The end desired in the present study is that the records of cold-period occurrence as to intensity, duration, and frequency shall be made available in simple and nonconfusing form for use in the study of ground freezing as affecting highway design. The method here outlined is offered for use as a practical means of accomplishing this end.

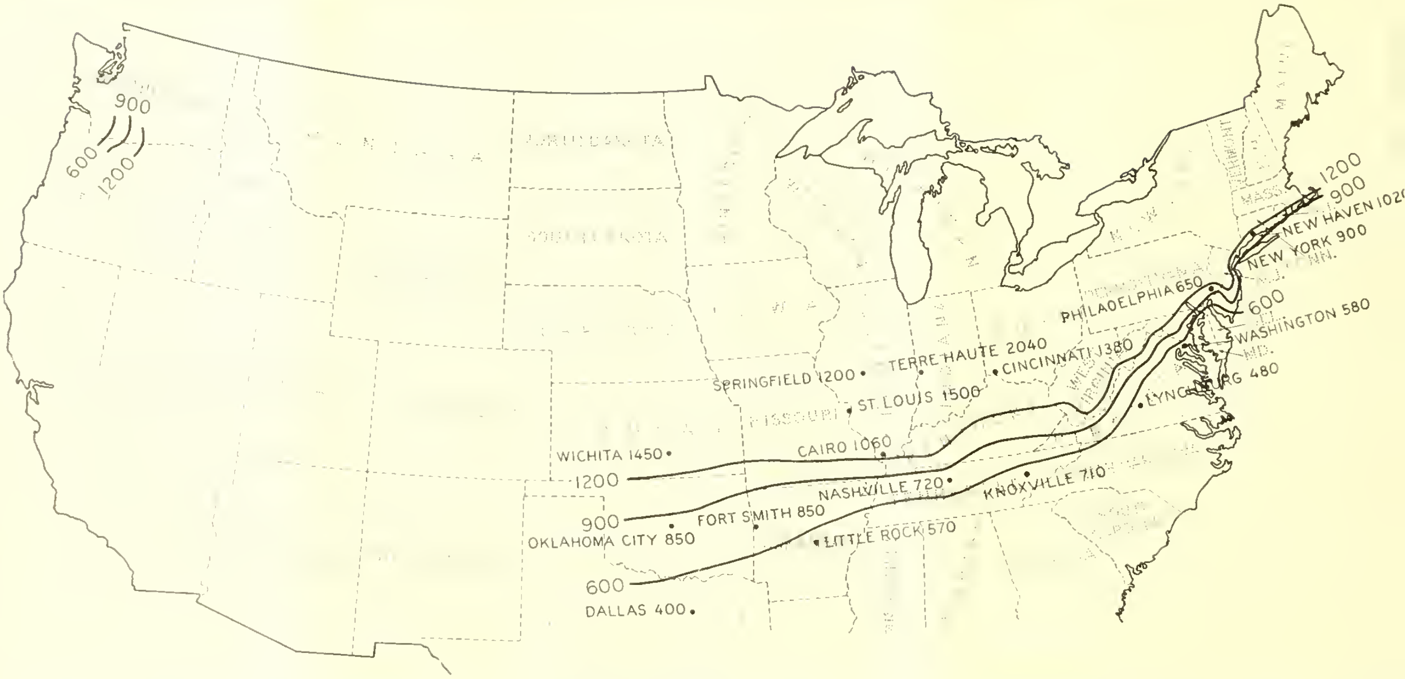


FIGURE 8.—ISOTHERMO-CHRONAL MAP OF THE UNITED STATES SHOWING LINES OF EQUAL DEGREE-HOURS OF COLD PERIOD OCCURRENCE (BELOW 23° F.)

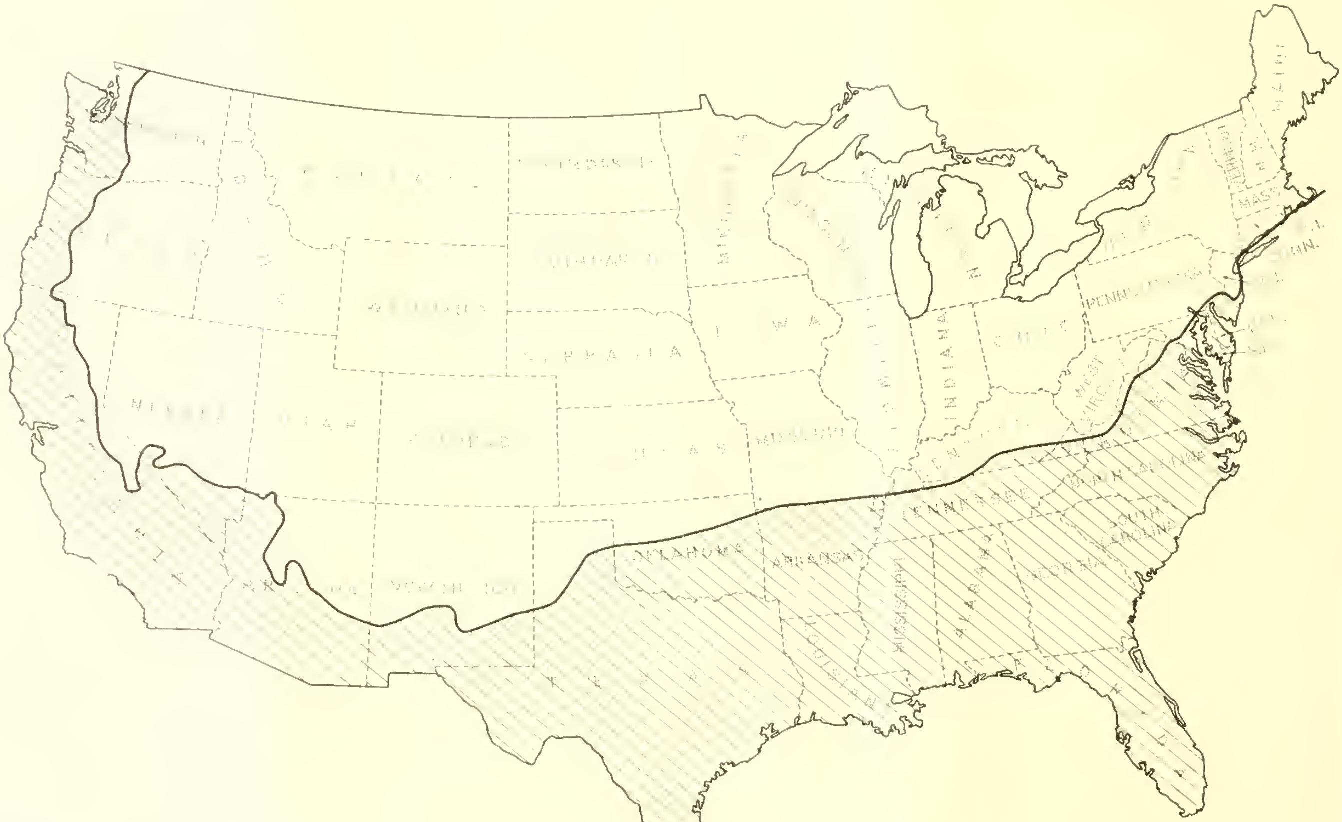


FIGURE 9.—MAP SHOWING CRITICAL INDEX LINE FOR HIGHWAY GROUND FREEZING, BASED ON MOST ADVERSE EXISTING CONDITIONS, AS SHOWN BY COMBINED STUDY OF FIGURES 7 AND 9

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UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

APRIL 30,1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	2,120.6	$ 2,004,449.68	$ 985,810.90	69.8	21.0	90.8	$ 824,724.63	$ 410,570.93	47.8	1.1	48.7	$ 4,066,796.43	Alabama
Arizona	767.9	4,159,473.73	3,135,588.24	137.5	175.6	313.1	130,751.34	98,442.65	.6	19.2	19.8	2,164,546.60	Arizona
Arkansas	1,726.0	5,530,742.55	2,622,939.34	143.9	33.5	177.4	1,375,360.75	689,581.83	60.7	18.9	79.6	2,173,046.38	Arkansas
California	1,885.4	7,283,127.24	2,929,074.69	157.5	24.9	182.4	639,997.57	354,314.85	9.8	16.2	26.0	2,782,525.77	California
Colorado	1,179.6	4,802,813.72	2,528,065.02	191.1	38.7	229.8	493,406.81	273,189.10	23.1	9.3	32.4	2,945,561.18	Colorado
Connecticut	240.3	2,090,867.97	940,716.39	7.9		7.9	865,678.07	241,903.04	13.8		13.8	778,000.92	Connecticut
Delaware	258.2	458,267.70	151,293.87	7.7		7.7	778,018.40	371,119.95	40.9		40.9	287,346.06	Delaware
Florida	474.3	5,567,349.78	2,578,887.73	126.3	5.5	131.8						1,782,889.95	Florida
Georgia	2,729.0	1,322,274.01	632,039.51	49.6	.8	60.4	815,165.49	401,875.29	42.4	5.0	47.4	4,810,859.46	Georgia
Idaho	1,193.9	854,241.38	515,387.33	64.3	27.8	92.1	437,011.84	264,237.30	24.5		24.5	1,729,939.70	Idaho
Illinois	2,034.6	15,384,518.19	5,781,398.40	433.3		433.3	3,115,515.07	1,247,835.84	62.2	38.0	100.2	6,785,639.24	Illinois
Indiana	1,344.0	7,884,191.46	3,703,856.95	245.0		246.0	1,795,791.91	830,079.97	55.6		55.6	2,387,368.46	Indiana
Iowa	3,156.7	516,581.32	258,995.00	19.1		19.1	7,653,136.07	3,252,334.85	58.7	204.7	263.4	1,892.77	Iowa
Kansas	2,787.2	4,645,497.37	2,150,681.89	250.5	10.3	270.8	1,025,380.95	510,678.44	55.2	10.7	65.9	2,848,480.89	Kansas
Kentucky	1,405.2	4,210,233.39	1,998,677.24	230.2	18.1	248.3						2,805,662.60	Kentucky
Louisiana	1,355.7	4,158,591.97	2,037,693.53	126.7	12.5	139.2	1,100,679.14	532,966.32	37.0	4.9	41.8	1,892,079.68	Louisiana
Maine	520.0	1,905,021.13	736,937.56	47.5		47.5	228,130.16	108,074.89	10.1		10.1	1,816,752.53	Maine
Maryland	630.7	1,403,037.45	674,833.10	58.3		58.3	1,070,109.23	430,439.09	22.2	12.6	34.8	480,939.45	Maryland
Massachusetts	657.6	3,194,615.51	979,971.48	43.8	2.6	46.4	479,176.44	205,634.08	11.7		11.7	2,561,258.16	Massachusetts
Michigan	1,612.8	9,840,644.44	4,191,186.45	222.2	30.5	252.7	200,000.00	100,000.00	11.1		11.1	3,359,523.89	Michigan
Minnesota	4,010.0	8,660,604.64	2,967,010.00	197.7	182.8	380.5	2,976,084.72	1,222,337.21	32.7	95.3	128.0	105,122.79	Minnesota
Mississippi	1,810.5	1,846,209.28	716,929.45	65.6	7.7	73.3	48,835.06	24,417.52	.1		.1	3,521,493.59	Mississippi
Missouri	2,445.8	6,989,724.78	2,522,488.79	98.2	68.9	167.1	6,265,936.81	2,105,808.89	94.8	61.2	156.0	1,138,426.58	Missouri
Montana	1,722.3	7,605,580.72	4,445,126.75	528.1	12.9	541.0	999,393.08	548,580.63	70.7	47.0	117.7	3,910,324.96	Montana
Nebraska	3,557.0	6,598,571.96	3,196,617.76	320.2	109.5	429.7	1,995,777.22	744,065.25	31.8	67.8	99.6	2,622,644.02	Nebraska
Nevada	1,160.0	1,139,782.79	1,012,696.90	72.0	109.3	181.3	320,691.26	284,934.17		32.9	32.9	1,075,347.59	Nevada
New Hampshire	350.6	496,706.30	152,386.14	5.5	2.1	7.6	130,463.20	44,724.92	3.5		3.5	607,316.42	New Hampshire
New Jersey	500.2	4,135,550.77	862,015.00	56.8		56.8	1,495,836.71	259,335.00	17.3		17.3	1,456,305.08	New Jersey
New Mexico	1,916.1	3,958,028.87	2,573,501.57	203.7	20.9	224.6	36,820.67	15,376.50	.8		.8	1,533,604.66	New Mexico
New York	2,458.2	18,480,217.76	3,787,505.00	253.2		253.2	8,055,650.00	1,519,650.00	101.5		101.5	8,217,060.99	New York
North Carolina	1,764.9	2,449,397.48	1,211,346.68	147.7	22.5	170.2	555,930.70	273,540.69	20.4		20.4	3,274,270.55	North Carolina
North Dakota	4,152.7	1,681,575.50	758,114.10	304.3	120.7	425.0	1,437,613.87	710,224.60	211.2	201.1	412.3	1,852,745.49	North Dakota
Ohio	2,178.5	14,739,816.88	4,495,146.34	255.0	18.8	273.8	7,696,513.53	2,484,111.08	133.4	21.4	154.8	2,683,163.25	Ohio
Oklahoma	1,904.3	2,525,942.07	1,086,589.78	68.6	29.8	98.4	2,064,986.06	900,879.15	56.9	32.2	89.1	1,677,751.44	Oklahoma
Oregon	1,147.7	3,098,574.37	1,867,346.97	187.8	66.2	244.0	2,541,838.63	1,430,849.49	66.3	29.2	95.5	708,673.78	Oregon
Pennsylvania	2,268.1	16,293,282.65	3,982,904.02	233.7	14.1	247.8	3,378,800.30	982,882.94	55.5		56.5	3,880,004.46	Pennsylvania
Rhode Island	184.8	1,884,585.72	534,553.58	27.8		27.8	129,559.02	64,779.50	.4		.4	657,590.02	Rhode Island
South Carolina	1,896.7	3,313,377.98	1,188,625.77	110.3	22.3	132.6	1,703,630.88	606,031.44	36.9	33.0	69.9	1,363,608.88	South Carolina
South Dakota	3,463.6	3,983,700.85	2,129,358.04	424.8	97.4	522.2	1,035,998.25	559,011.83	98.1	79.7	177.8	1,258,005.23	South Dakota
Tennessee	1,243.3	1,871,857.29	817,145.16	74.8		74.8	863,957.19	431,978.59	26.7	12.5	39.2	3,143,092.20	Tennessee
Texas	6,786.6	11,042,254.40	4,467,528.41	330.5	84.8	415.3	4,239,420.02	1,629,901.44	143.1	79.5	222.6	5,735,367.72	Texas
Utah	988.9	752,789.23	502,869.44	31.8		31.6	670,819.20	490,941.75	28.4	89.6	118.0	1,010,284.26	Utah
Vermont	255.2	1,909,459.07	525,508.11	34.5	2.6	37.1	575,492.53	170,919.86	10.2		10.2	186,919.47	Vermont
Virginia	1,417.6	4,234,953.10	1,982,279.12	174.3	23.3	197.6	1,138,964.99	551,746.88	39.0	2.2	41.2	906,952.51	Virginia
Washington	925.8	2,911,647.90	1,132,300.00	55.6	40.2	95.8	1,302,420.65	507,000.00	45.0		45.0	1,538,673.29	Washington
West Virginia	717.1	2,842,201.32	1,136,062.75	69.1	12.4	81.5	1,054,295.45	468,023.81	15.3	21.6	36.8	885,321.29	West Virginia
Wisconsin	2,265.5	2,985,320.49	1,303,185.13	114.5	1.7	116.2	3,144,764.00	1,256,500.00	58.0	44.6	102.8	1,893,186.87	Wisconsin
Wyoming	1,767.1	1,333,730.81	862,873.54	118.6	24.3	142.9	266,012.65	174,637.77	20.1	43.9	64.0	1,405,545.48	Wyoming
Hawaii	39.6	964,949.48	384,686.43	23.2		23.2						1,682,039.16	Hawaii
TOTALS	83,475.3	228,058,133.45	93,139,625.55	7,229.9	1,487.0	8,716.9	79,155,631.58	30,686,469.33	2,006.3	1,335.2	3,341.5	108,464,160.05	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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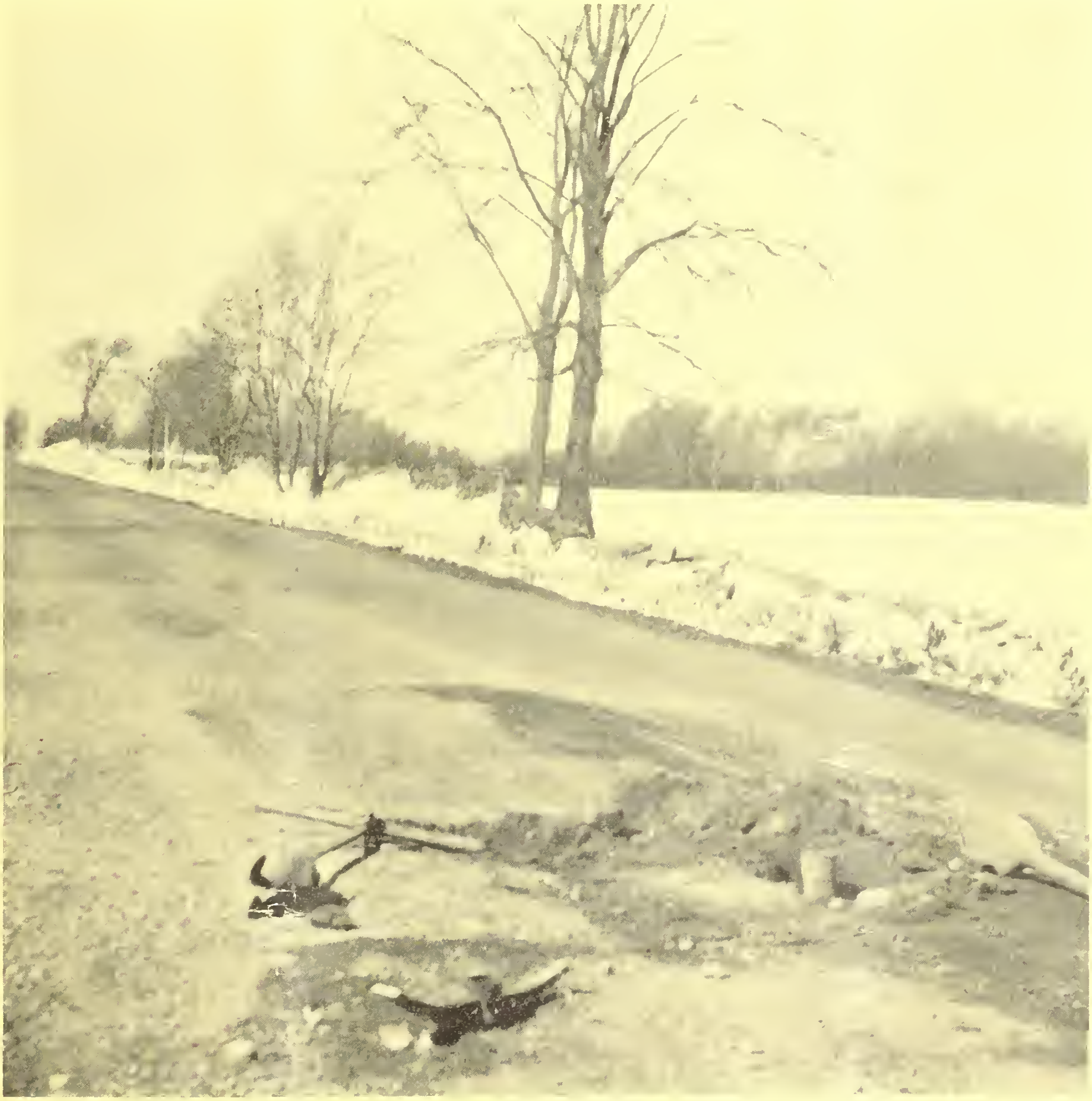
A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 4 ▽ JUNE, 1930



HIGHWAY DAMAGED BY GROUND FREEZING

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 11, No. 4 **JUNE, 1930** **R. E. ROYALL, Editor**

TABLE OF CONTENTS

Page

Illustrations of Frost and Ice Phenomena 61

Progress Report of the Connecticut Avenue Experimental Road 69

Some Points of Contact Between Soil Science and Highway Engineering 77

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ILLUSTRATIONS OF FROST AND ICE PHENOMENA

Reported by **IRA B. MULLIS**, Associate Engineer of Tests, Division of Tests. United States Bureau of Public Roads

THE heaving of soils due to frost action and the subsequent loss of soil stability due to thaws constitutes one of the most important problems confronting both the highway and railroad engineer. The purpose of this report is to call attention to the many different ways in which frost action may manifest itself. As a basis for the discussion to follow, the physical laws controlling the performance of soils during frost action are presented in summarized form.

PHYSICAL LAWS CONTROLLING EXPANSION DUE TO FROST ACTION REVIEWED

Failures due to frost action may be due to physical phenomena occurring either individually or in combination with each other as follows: (a) The gradual expansion of freezing water; (b) the instantaneous freezing of supercooled water when the pressure productive of supercooling is removed; (c) the contraction and expansion of either ice or frozen soil due to temperature changes; and (d) the growth of ice layers in moist or wet freezing soils.

Expansion of freezing water.—Water at any temperature in excess of 4° C. (39.2° F.) expands when heated and contracts when cooled. It attains its maximum density at 4° C., however, and cooling below this temperature causes the water to expand. The rate of this expansion *(1)*[1] is such as to cause water at decreasing temperatures to possess relative volumes as follows: 4° C., 1.00000; 0° C., 1.00013; −5° C., 1.00070; −10° C., 1.00186. The corresponding densities are: 4° C., 1.00000; 0° C., 0.99987; −5° C., 0.99930; and −10° C., 0.99815. It is understood, of course, that unfrozen water at or below 0° C. can exist only when the water is under pressure.

The expansion which water undergoes on changing from the liquid to the solid state without change in temperature exceeds very appreciably any volume change due solely to change in the temperature of the water. This expansion amounts to 9 per cent of the initial volume *(2)*. Thus 100 cubic feet of water at 0° C. may become 109 cubic feet of ice at the same temperature.

Freezing of supercooled water.—When the expansion of water due to cooling is prevented, the freezing point is lowered and the water exerts very high pressures. The interrelationship existing between the freezing point of water and the pressure under which the water exists, according to Bridgman *(3)*, is shown graphically in Figure 1. According to this figure, supercooling to −4° C. causes the water to exert a pressure of about 6,500 pounds per square inch. This illustrates the tremendously high pressures exerted when supercooled water is not permitted to expand.

When the pressure exerted by the water exceeds the resistance of the container, freezing occurs at a rate dependent upon the speed at which the pressure is released. At the instant when solidification occurs the volume of the contained material increases 9 per cent.

Thus when the container consists of plastic materials such as thin lead pipe, partly frozen soil, etc., the release of pressure may occur gradually. But in a container formed of cast iron, ice, or solidly frozen soil, the pressure release will occur suddenly, and thus cause the ice to form in a more or less explosive manner.

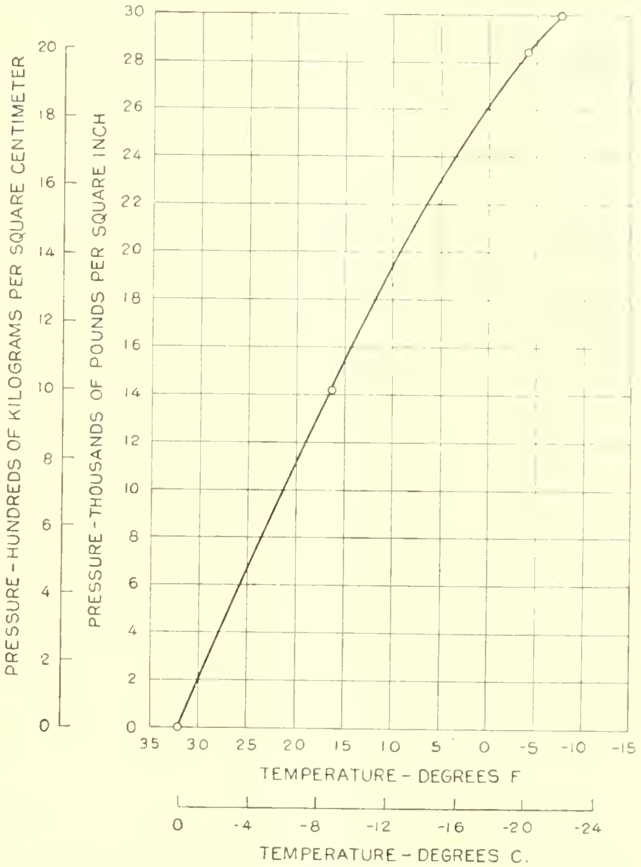


FIGURE 1.—RELATION BETWEEN THE FREEZING POINT OF WATER AND THE PRESSURE UNDER WHICH THE WATER EXISTS. DATA FROM SMITHSONIAN PHYSICAL TABLES (1921)

This explosive action of ice is illustrated by experiments by Major Williams *(4)* and described as follows:

> Having quite filled a 13-inch iron bombshell with water he firmly closed the touchhole with an iron plug weighing 3 pounds and exposed it in this state to the frost. After some time the iron plug was forced out with a loud explosion, and thrown to a distance of 415 feet, and a cylinder of ice 8 inches long issued from the opening. In another case the shell burst before the plug was driven out, and in this case a sheet of ice spread out all round the crack. It is probable that under the great pressure some of the water still remained liquid up to the time at which the resistance was overcome; that it then issued from the shell in a liquid state, but at a temperature below 0° C. (32° F.), and therefore instantly began to solidify when the pressure was removed and thus retained the shape of the orifice whence it issued.

The cake of ice furnished by J. L. Harrison, of this bureau, and shown from two different angles in Figure 2 also illustrates the instantaneous solidification of supercooled water. The water which formed this cake was but partly frozen in an aluminum pan. When photographed the cake contained both free water and air. Due to the high degree of heat conductivity possessed by aluminum, freezing probably began simultaneously at the top, bottom, and sides of the water mass, thus inclosing the unfrozen water in an ice container. As the process of freezing continued, the contained water exerted increasing pressure until the resistance of the ice container was exceeded. The pressure being suddenly released, the supercooled water immediately solidified and the expansion due to freezing caused the protruding "tooth" to be formed as shown.

---

[1] Figures in parentheses refer to reports listed in the bibliography at the end of this report.

The bulge in the top of the ice as shown in Figure 2 (bottom) is believed to be due to the expansion of solidifying water when confined by the surface crust of ice. This bulge was highest at the center and did not extend quite to the circumference of the cake. Fissures concentric with the circumference of the cake were observed along the circumference of the bulge in the bottom of the top crust of ice where cross bending was greatest. Apparently the supercooled water burst through one of these fissures to form the "tooth" or cone of ice shown.

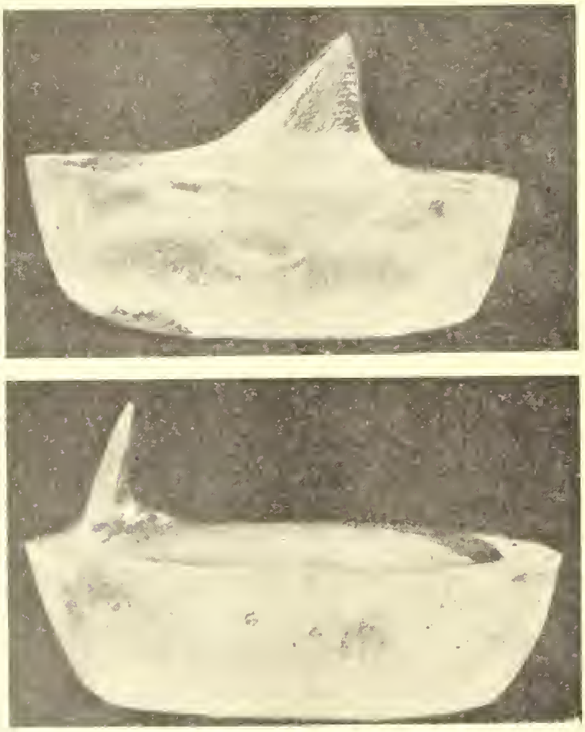


Figure 2.—A Cake of Ice Which Froze on All Sides but Burst Under the Expansive Force of the Freezing Water in the Interior

Volume change due to temperature change occurring in ice.—According to tests performed by Petterson (*5*) the coefficient of linear expansion of ice is 0.000053 per degree centigrade. Therefore if the temperature of an ice cake 500 feet long were reduced from 0° C. to −18° C., its length would be reduced by 0.477 foot. Should this contraction result in cracks which are subsequently closed by newly formed ice, the length of the ice cake when returning to a temperature of 0° C. becomes 500.477 feet. With succeeding temperature alternations of this character, the length of the ice sheet would continue to increase. The effect of these temperature alternations on the growth of the ice is analogous to the strokes of a jack handle in moving loads. As the magnitude of the work done by the jack is governed by the length and the number of strokes of its handle, so is the growth of the ice controlled by the amplitude and the number of temperature variations below the freezing point of water.

Growth of ice layers in soil.—According to both Taber (*6*) and Bouyoucos (*7*) the formation of well-defined ice layers in freezing soil is due to three physical phenomena: (*a*) The ability of water particles in soil pores of comparatively large capillary dimension to freeze at or slightly below normal freezing temperature; (b) the ability of water particles in the finer capillaries to resist freezing until the temperature is reduced to a point below that at which the moisture froze in the larger capillary pores; and (c) the ability of water particles when freezing in the larger capillaries to draw to themselves unfrozen water from the finer capillary pores and thus increase in volume at the expense of the water furnished by the finer capillaries. Figure 3 illustrates the segregated ice layers in a cylinder frozen, by Taber.

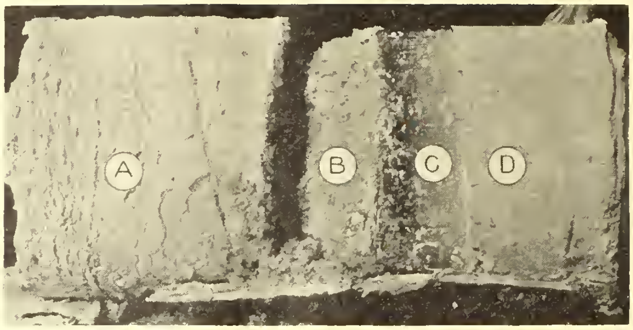


Figure 3.—Frozen Clay Cylinder with Sand Layer in Middle. A, Frozen Clay; B, Sand; C, Ice; D, Unfrozen Clay. From Report by Taber (*6*)

Pressure effects due to frost action.—Both Taber and Bouyoucos state that enormous pressures are produced by ice crystals during growth. According to Taber the growing ice crystals may produce pressures in excess of 14 kilograms per square centimeter (199 pounds per square inch) and may cause water to be placed under a tension sufficient to lift a column of water over 150 meters (492 feet) in length.

The magnitude of the pressures exerted by expanding ice also is very high. The horizontal thrusts exerted upon dams and other structures due to this cause have been estimated to be as large as 34,000 to 47,000 pounds per lineal foot. Compressive tests (*5*) made on blocks of ice by Professor Brown of McGill University showed strengths as follows:

Temperature °F.	Crushing strength, pounds per square inch
28°	300
14°	693
2°	811

If the crushing strength of ice equals 400 pounds per square inch, as assumed by C. A. Mees in his paper on the design of dams (*5*), the thrusts exerted by various thicknesses of ice forming on the surfaces of lakes or similar bodies of water are as follows:

Ice thickness, inches	Horizontal thrust in pounds per lineal foot
6	28, 800
8	38, 400
10	48, 000
12	57, 600

But when the temperature of the ice approaches 0° F. the above values probably double.

VARIABLES FACTORS INFLUENCE THE MANIFESTATION OF FROST ACTION

Although the physical laws which control the volume increase of solidified water may be stated simply, the particular manner in which the effects of frost action are manifested depends upon a number of variables, among which are: Direction of heat radiation; size of soil particle; and amount of water available. Generally, the vertical manifestation of frost action is termed heave and the horizontal manifestation is termed thrust.

Direction of heat radiation.—According to Taber (*6*) the upward heave that accompanies the freezing of soils is due to the growth of ice crystals in a vertical direction, and this is determined by the direction in which heat is conducted away most rapidly and the availability of water necessary for growth. The importance of the direction of heat conduction is illustrated in the experiments described below.

Mixtures of white clay and water in different proportions were frozen in thin glass test tubes, half of them being buried in sand so that freezing was from the top down, while the others were exposed so that freezing took place from the sides inward. All of the latter were broken, longitudinal cracks extending the full length of the test tubes; but where frozen from top down none were broken, for the ice crystals grew only in a vertical direction.

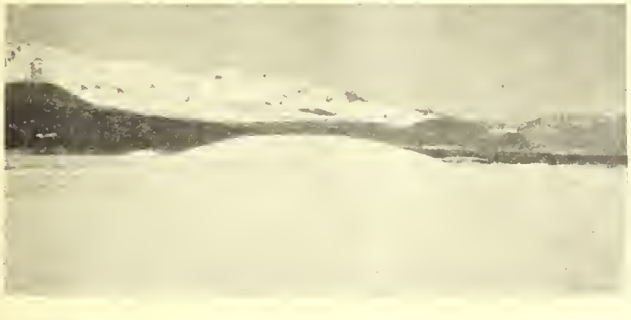


FIGURE 4.—AN ICE HEAVE OR MOUND COMMON ON FROZEN RIVERS IN ALASKA (PHOTO BY U. S. G. S.)

Size of soil particle.—In coarse-grained sands free to drain no important frost heaves occur because practically all of the contained water freezes at normal freezing temperature and small unfrozen water particles do not exist in amounts sufficient to cause the frozen particles to suffer appreciable growth.

Permeable silts, which are capable of raising water rapidly and through considerable distances, are apt to suffer considerable frost heave.

The capillary tension may be higher in cohesive clays than in silts. The speed with which water rises in clays, however, is much less than in silts. Consequently, in dense clay soils with low ground-water level and absence of lateral seepage, only limited amounts of water are available for ice segregation. Under these conditions the soil adjacent to the growing ice crystals is apt to dry out and shrink, due to the loss of moisture. The ground-water elevation in clays must be comparatively high in order that much frost heave may occur or the clay must be wet, due to water absorption from the top of the ground.

Amount of water available.—According to Taber, high water content favors segregation and additional water may be drawn from the water table to form very thick ice layers. Studies made by Eakin (*8*) in Alaska indicated that—

> In materials which favor even distribution of water throughout the mass, heave is uniform over the entire surface and no differential vertical movement occurs. Thus, in fine, even-grained materials horizontal movement or movement with the surface is dominant, and even surfaces, either horizontal or sloping, result. On the other hand, irregular capacity for the retention of interstitial water leads to differential heave and thrust and to the development of surface irregularities.

These statements seem obvious, but differential heave also occurs in fine even-grained materials where the several areas receive quantities of water in varying amounts just prior to the period of ground freezing. The differential heaving in this case, however, is not generally so pronounced as in soils possessing nonuniform capacities.

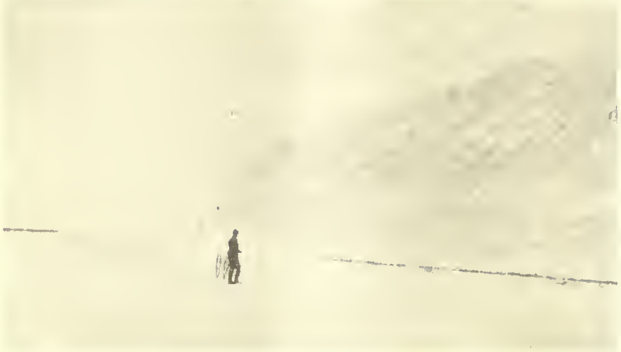


FIGURE 5.—SHOWING THE RELATIVE HEIGHT OF AN ICE HEAVE OR MOUND ON A FROZEN RIVER IN ALASKA. (PHOTO BY U. S. G. S.)

Furthermore, it should be remembered that when lakes or similar bodies of water are of such depth that water at the bottom is not reduced below 4° C. no freezing takes place at the surface even when the air temperature is considerably below 0° C. For this reason the surface of the deeper portions of deep bodies of water such as Hudson Bay and Lake Ontario have not been observed to freeze within historical time (*5*).

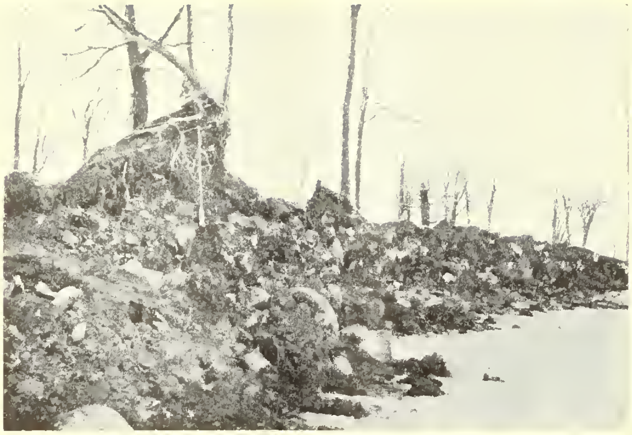
DETRIMENTAL FROST PHENOMENA ILLUSTRATED

The natural manifestations of frost action are necessarily varied in character because of the number of conditions under which the phenomena are apt to occur. Furthermore, the phenomena occurring in some complex combination instead of individually are probably responsible for detrimental pavement heaving or other damage. Under all conditions, however, the occurrence of these phenomena furnishes evidence of the enormous force exerted by freezing water. This is illustrated by the mounds, ramparts, frost boils, etc., referred to in the following discussion.

Figures 4 and 5 illustrate ice mounds familiar to those who travel in polar and subpolar regions when the rivers are frozen. The occurrence of these mounds is explained as follows:

With the beginning of freezing weather ice forms along the banks of the stream and becomes firmly attached to the soil and rocks located there. When the ice sheet becomes continuous from bank to bank and gradually grows thicker the flow channel becomes correspondingly smaller. Under these conditions the water beneath is apt to be compressed until the force caused by this compression causes the surface ice to heave at the weaker areas. Water frequently spouts through the fissures which often form about these heaves, and attendant flooding and freezing continues throughout the winter or until the volume of water is reduced to such amount that it may be contained beneath the ice. These pressures sometimes become so great that water is forced out through the banks between rock strata or other openings and quickly freezes.

Figure 6 shows a bank of soil composed of sod, bowlders and clay which, according to Doctor Buckley (*9*), was thrown up during the winter of 1898–99 by ice thrust on the edge of a lake in Wisconsin. The average dimensions of this bank were a height of about 4 feet, a breadth of base of about 11 feet, and a breadth of top



Courtesy Wisconsin Academy of Sciences, Arts and Letters

Figure 6.—The Result of Ice Thrusts on Picnic Point Near Madison, Wis. (Photo by Doctor E. R. Buckley)

of about 4 feet. At one place this bank had a height of not less than 8 feet and had raised a tree of considerable size through this distance. (Fig. 6.) Doctor Buckley says that bowlders were in many places actually rammed into the bank in such a manner that they presented much the appearance of plums in a pudding. The bank, in many places vertical, was raised up and turned over by the ice shove and trees 12 inches or more in diameter were sometimes dislodged and moved.

The cause of ice ramparts has been quite clearly described by Gilbert (*10*) as follows:

> The ice on the surface of a lake expands while forming, so as to crowd its edge upon the shore. A further lowering of temperature produces contraction, and this ordinarily results in opening vertical fissures. These admit water from below and by the freezing of that water they are filled, so that when expansion follows a subsequent rise of temperature, the ice can not assume its original position. It consequently increases its total area and exerts a second thrust upon the shore. Where the shore is abrupt, the ice itself yields, either by crushing at the margins or by the formation of anticlines elsewhere; but if the shore is generally shelving, the margin of the ice is forced up the acclivity, and carries with it any bowlders or other loose material about which it may have frozen. A second lowering of the temperature does not withdraw the protruded ice margin, but initiates other cracks and leads to a repetition of the shoreward thrust. The process is repeated from time to time during the winter, but ceases with the melting of the ice in the spring.

OCCURRENCE OF FROST CRYSTALS AND FROST BOILS DISCUSSED

Figure 7 illustrates the occurrence of frost crystals and thin ice sheets frequently observed on clay roads which are more or less rutted. When the temperature of the ground surface reaches the freezing point, ice crystals form on the surface of moist soil, and ice forms on the surface of any pools of water which may exist on the surface. Following these freezing temperatures one frequently notices that these recently water-filled depressions are covered with thin sheets of ice with little or no water underneath. Upon further examination it will be noted that all fissures or other soil openings contain ice crystals. The absence of water underneath ice-covered depressions is due to the water having been withdrawn from under the ice sheets by the increased capillary tension of the soil produced by lowered temperature. It will be noted that ice crystals on the surface of clay soils will be longer than on those soils which contain less clay. Compacted clay soils will, however, contain fewer frost crystals at the surface than a similar soil in a less compacted state.

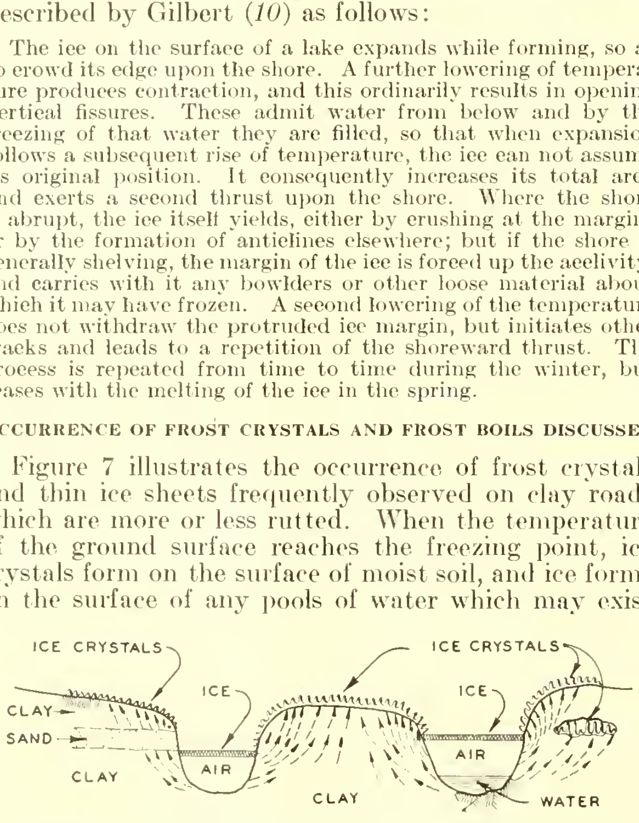


Figure 7.—Illustrating Water Movements, Frost and Ice Formations on a Wet Rutted Clay Surface

Should a layer of relatively dry sand exist somewhat below the surface as illustrated on the left side of Figure 7, the ice crystals are apt to be much shorter if present at all. It seems that the conditions essential for producing crystals of maximum length are found where the intensity of cold is such that the capillary movement is rapid enough to prevent freezing to an appreciable depth beneath the surface. This type of freezing is illustrated by Figure 8 which shows the growth of ice crystals during three consecutive nights when there was little or no thawing during the intervening days. Had there been a considerable drop in the air temperature in the early part of the third night of freezing, the height of the ice crystals in the lowest stratum (fig. 8) would have been proportionatly reduced and a frozen stratum of wet soil would have resulted.

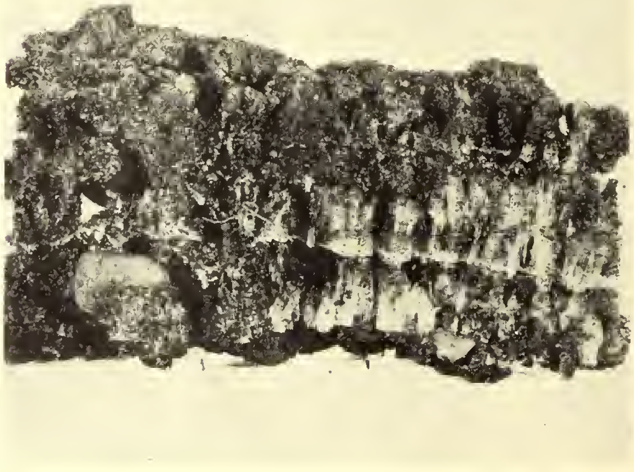


Figure 8.—Showing Three Stages of Growth of Ice Crystals Formed on the Surface of Wet Clay During Three Consecutive Nights of Freezing Temperature with Little or No Thawing During the Intervening Days

Figure 9 shows the surface appearance of a "frost boil" under a surface-treated macadam road in New Hampshire. The origin of the water in the boil was traced to a leaking water pipe beneath the subgrade. The type of soil was a fine silt which served as a water pocket. Whether this heaving was produced by solid ice or by ice crystals which had become more or less segregated from the soil by frost action is not known. It seems probable, however, that much if not all of this heaving was due largely to a mixture of ice and soil of a form somewhat like that shown in Figure 8.

Figure 10 shows solid ice overlaid by a few inches of thawed material underlying a concrete pavement near Duluth, Minn. When first observed this frost heave must have extended 8 or 9 inches above the original grade line. The concrete pavement although heavily reinforced with longitudinal steel bars was badly shattered. An excavation made at one edge of the pavement disclosed that the pavement was laid on a bed of sand

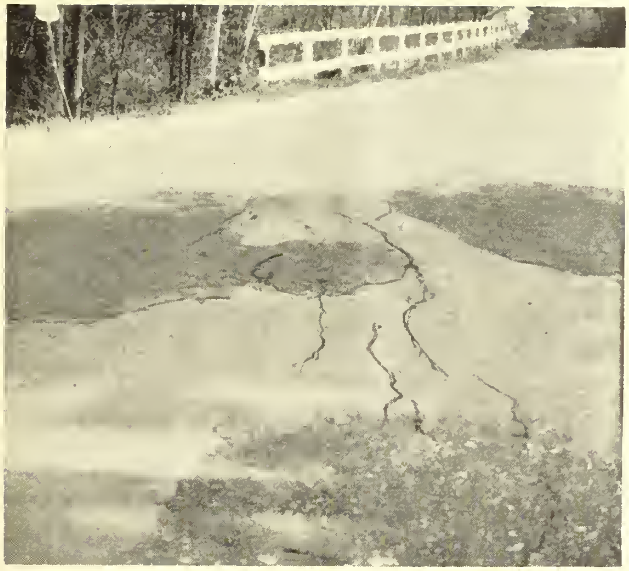


Figure 9.—A Frost Boil in a Surface-Treated Macadam Laid on Bed of Silt. The Heave was Due to a Leaking Water Pipe Beneath Followed by Freezing

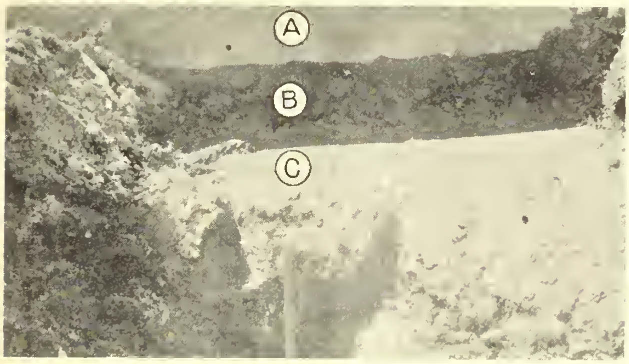


Figure 10.—Photograph of the Lower Part of a Ruptured Concrete Pavement at A; B, the Thawed Subgrade; C, the Underlying Prism of Solid Ice and Frozen Clay Underneath

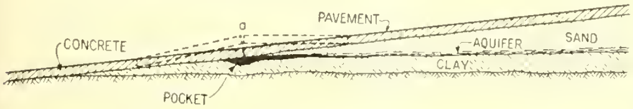
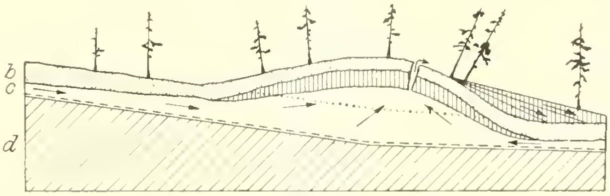


Figure 11.—Subgrade Profile Productive of Frost Heaves

underlaid by a stratum of clay. Where the dipping stratum of clay was intersected by the plane of the subgrade a water pocket formed as is illustrated in Figure 11. Clay shoulders in the sides and a formation of solid ice beneath (fig. 10) prevented the escape of the water from this pocket.

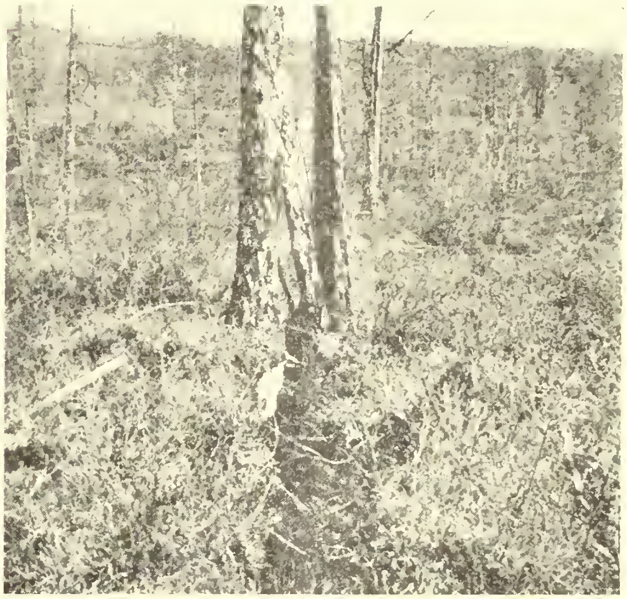
Figure 12 illustrates the occurrence of soil blisters. According to Nikiforoff, (*11*) these blisters attain enormous proportions in Arctic regions, extending more than 20 feet in height in Siberia. The method of formation is explained as follows: *d* represents ever-frozen subsoil which extends to considerable depths. The soil above this stratum thaws during the summer and as is characteristic of most Arctic regions is frequently quite wet, especially near the foot of slopes and in surface depressions where water collects. At the beginning of winter the surface soil, *b*, freezes down to the water in liquid form *c*. At this stage the liquid water is confined between two zones of hard-frozen material. When the pressure becomes sufficiently high to rupture the frozen surface soil, the supercooled water is forced through the crack and is probably frozen instantly. Figure 13 shows a tree split by a fissure occurring on the summit of an ice blister.

Figure 14 illustrates similar mounds formed of more or less segregated fragments and particles of alluvial materials believed to be of Pleistocene or recent age observed by Leffingwell (*12*) and others in Alaska. Most of these mounds are in the form of gentle domes ranging in height from less than 25 feet to a maximum of about 200 feet above the plain. Generally they have rounded tops and slopes of less than 15° from the horizontal but a very few are steepsided and have an angular break at a somewhat level top. Some of the more recent mounds contain craters which furnish fresh water to such extent during the summer that they may overflow.



Courtesy Soil Science

Figure 12.—Cross-Section of a Soil "Blister" in Siberia



Courtesy Soil Science

Figure 13.—Tamarack Tree on the Summit of a Blister, Split When the Blister Broke

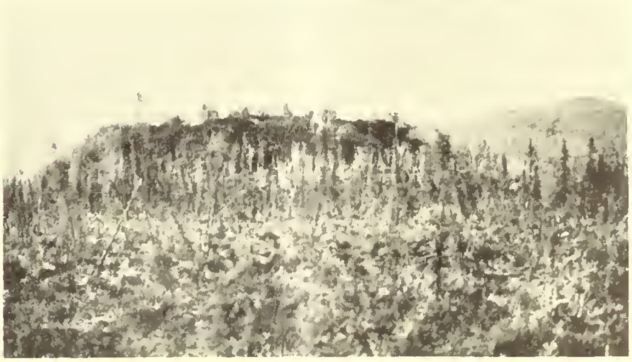


Figure 14.—A Mound Formed of Mud and Gravel Believed to Have Formed from Springs Subjected to Frost Action. (Photo by U. S. G. S.)

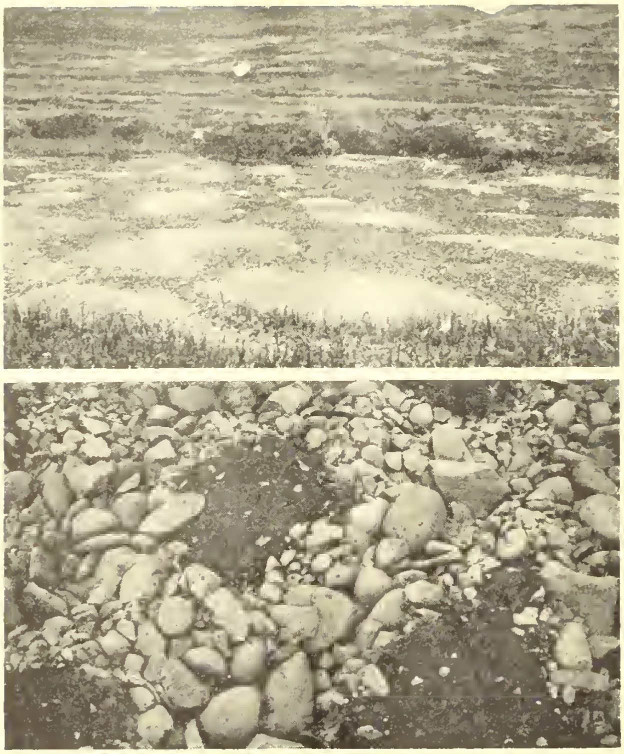


Figure 15.—A Frost-Heaved Area at the Foot of a Slope in the Lake Clark-Kuskuskokwim Region, Alaska, and a Close Up of One of the Frost Heaves. (Photos by U. S. G. S.)

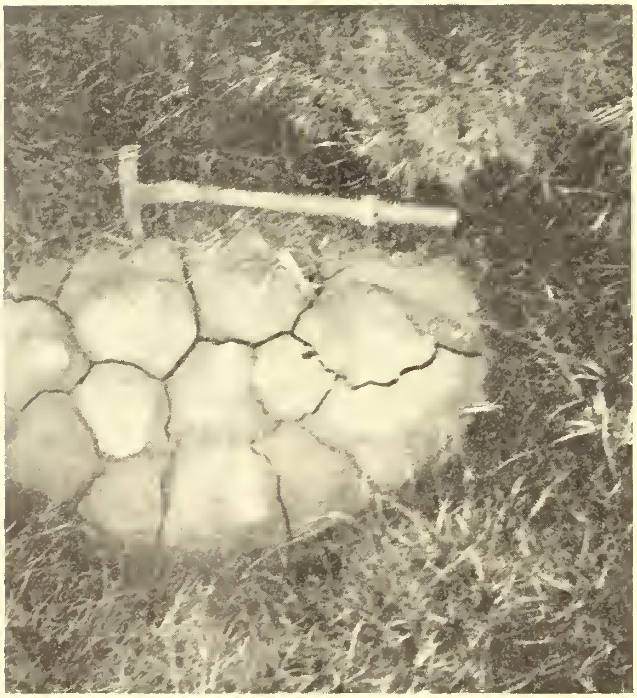


Figure 16.—A Dried and Cracked Mud Boil Found in Alaska by Eakin. (Photo by U. S. G. S.)

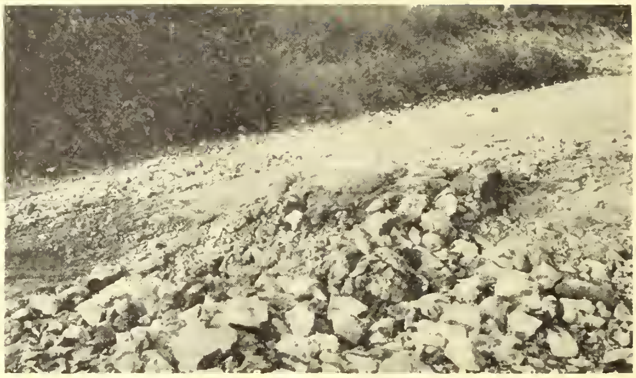


Figure 17.—A Mud Boil in a Water-Bound Macadam Laid on a Fine Sandy Clay Soil

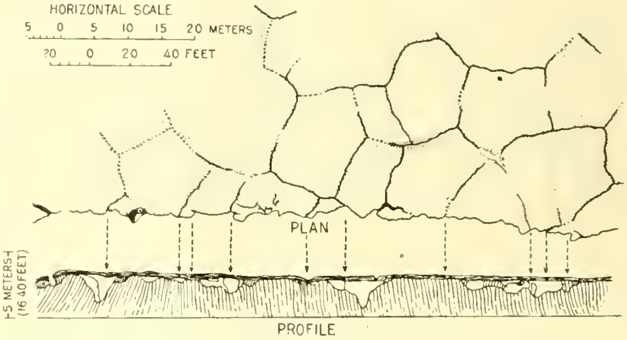


Figure 18.—Map of Frost Cracks on the Tundra, with a Sketch of the Exposures of Ground Ice in the Bank at One Edge of the Mapped Area. (From Report of U. S. G. S.)

Figure 15 (upper picture) shows a frost-heave area in the foreground at the foot of a slope in Alaska. These mounds, while small in comparison to some of those just described, seem to occur under conditions more or less similar to those productive of the larger ones. Where the development has progressed beyond the youthful stage these frost boils are invariably largely made up of mud at the center with the particles increasing in size toward the edge where stone and gravel predominate.

Figure 15 (lower picture) is a close-up view of one of these heaves. While geologists do not seem to be in full accord as to the exact manner in which this segregation of the finer particles from the larger rock fragments occurs, they are agreed that in all cases these mounds occur only where the ground is more or less thoroughly saturated with water and is subjected to conditions of alternate freezing and thawing of the surface layers of the ground (*8*, *13*).

Figure 16 shows the appearance of a dried and cracked mud boil found in Alaska by Eakin (*7*), which probably originated under the type of frost action just described.

The only difference between frost and mud boils is the manner in which the water accumulates. In reality discharging frost boils are nothing more than mud boils formed under frost action. Figure 17 shows a mud boil found about the middle of summer in the District of Columbia on the surface of a water-bound macadam laid on a fine sandy clay soil. This mud boil originated from a leaking water pipe and is similar in detail to the frost boils already described. It was first observed as a small mud-filled fissure on the road surface. With each passing load an eruption occurred and the volume of mud and water at the surface increased until a considerable mound was formed. Such boils may occur in any region where very fine particles of soil become reduced to the liquid state and under pressure the viscous fluid is extruded through fissures or other openings of such size as will permit its passage.

Figure 18 illustrates fissures formed when frozen soil in Alaska contracts due to a lowering of the temperature. According to Leffingwell (*12*) this cracking

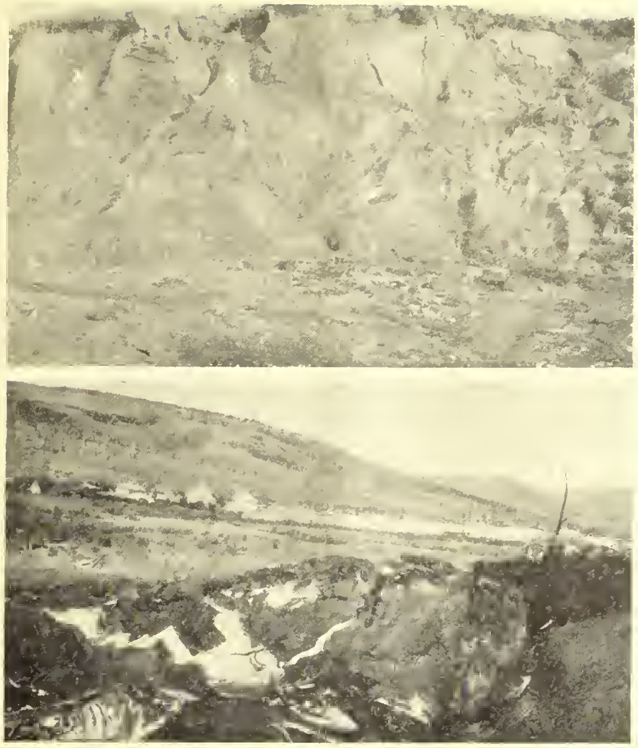


Figure 19.—Ice Wedges in Frozen Soil in Alaska. (Photo by U. S. G. S.)

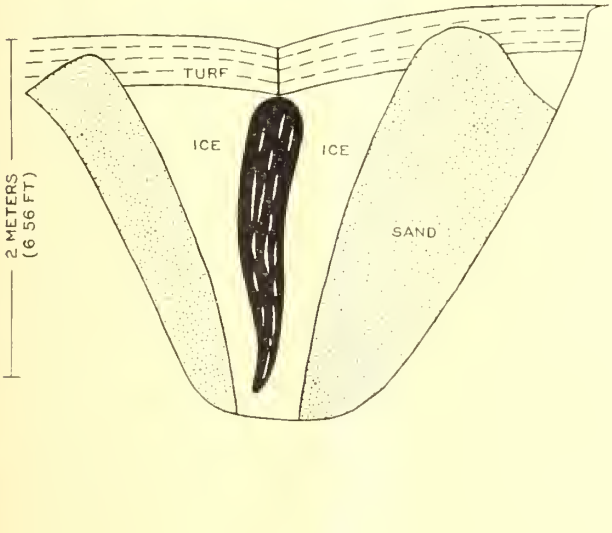


Figure 20.—Ice Wedge in Sand (Upper Picture), Showing Tunnel Cut by Drainage of Water Through Frost Crack, and in a Muck Bed (Lower Picture), Showing Upturned Strata. (From Report of U.S.G.S.)

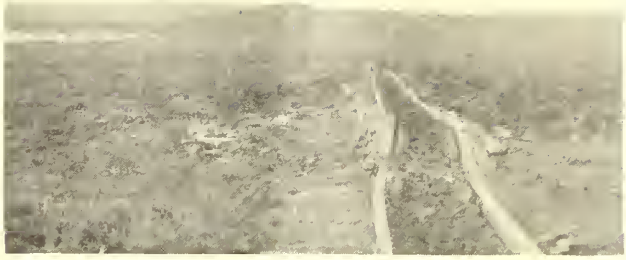


Figure 21.—Effect of Heave and Thrust on the Position of a Tram Line in Alaska. (Photo by U. S. G. S.)

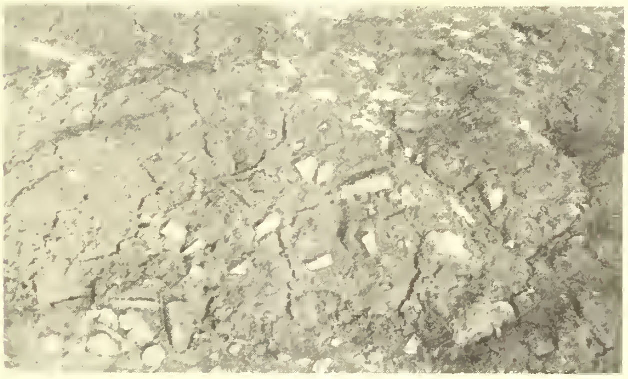


Figure 22.—Shrinkage Fissures in Frozen Clay Soil in the District of Columbia

is frequently accompanied by loud reports and shocks sufficient in intensity to rattle dishes in camps. The individual blocks formed by these cracks have an estimated average diameter of about 16 yards. When the snow melts in the summer these cracks can be seen cutting across all tundra formations and even across the mud and the growing moss beds. Somewhat similar phenomenon has also been observed in Siberia by Nikiforoff (*11*).

Figure 19 illustrates ice wedges shown by X which are apt to form due to the cracking described above. The snow which fills the newly formed fissures soon becomes compacted and may reach a density equal to that of solid ice under conditions of alternate freezing and thawing such as frequently occur toward the end of the polar winters.

The disturbance of the adjoining soil due to formation of ice wedges is illustrated in Figure 20. When these wedges occur along the banks of streams and lakes large masses of earth are frequently displaced under wave or current action, causing the banks to have a saw-tooth appearance. The displacement of the tram line illustrated in Figure 21 is attributed at least in part to the formation of ice wedges.

Figure 22 illustrates fissures observed in frozen clay soil in the District of Columbia. Similar fissures ranging in width from one-eighth to one-fourth of an inch were observed also in Minnesota. In both cases the fissures contained hoar frost, but no ice crystals were observed. Hoar frost is always derived from water vapor but ice crystals such as those shown in Figure 8 are always formed from unfrozen moisture at the base of the crystals.

Figure 23 illustrates the type of soil migration which may occur when snowfall is heavy and frost penetrates the ground to an appreciable depth.

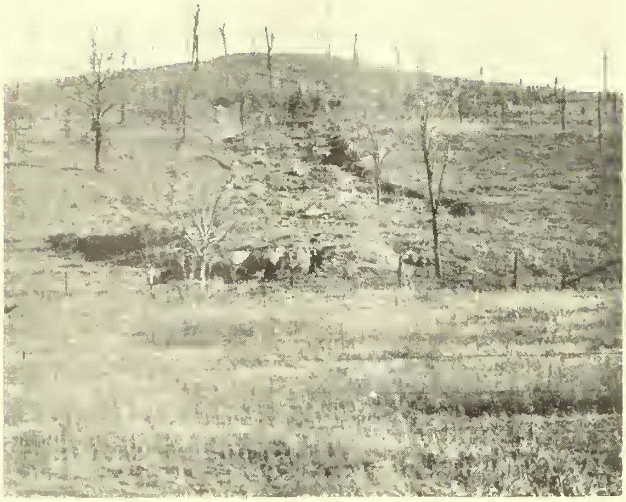


Figure 23.—An Avalanche of Mud, Soil, and Rocks Which has Slid Down a Hillside of Shaley Soil

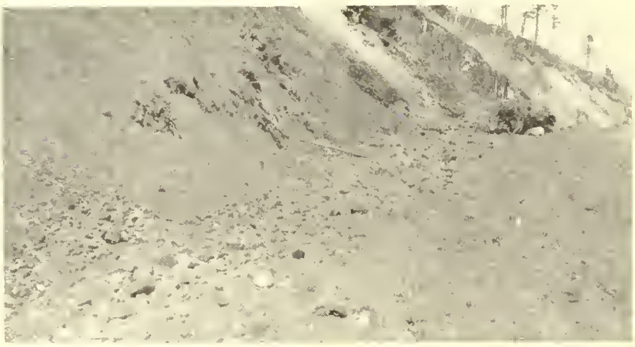


Figure 24.—Effect of Frost on Soil Migration Where the Particles Range from Fine to Coarse and Are Not Compact in Arrangement

During the summer in polar and subpolar regions the surface of the frozen earth, which has become more or less thoroughly broken up under frost action during the long winter, is in a state favorable to becoming thoroughly soaked by water from melting snow and ice. Under this condition masses of viscous fluid of varying magnitude flow down most of the slopes and in certain cases fill the streams with great volumes of mud, ice, and water. Almost everywhere in polar regions where there are heavy deposits of snow, this type of soil movement is markedly present where long periods of alternate freezing and thawing occur (*14*, *15*).

Figure 24 shows the type of soil migration from the face of cuts in porous soils after frost action.

Differential heaving underneath road surfaces and pavements seems to occur when the subgrade soil possesses variable capacity for the retention of ground water. Coupled with this, there must be channels for furnishing water to the locations where the heaves occur.

Many investigators have reported the occurrence of frost heaves over water pockets similar to those shown in this paper. Arnold (*16*) found frost heaves in both cuts and fills under surface-treated macadam roads. Those in cuts were found to be due to seepage from contiguous slopes while those on fills were sometimes due to entrapped water flowing through the porous stone base from wet cuts at the upper ends of fills. Frost heaves may very readily occur on fills consisting of materials possessing both water capacity and permeability different in amounts.

According to Moffitt (*17*) most of the ice beds in Alaska occur either in the deposits of silt or between the beds of silt and the underlying gravel. Veins of ice in some places cut across the beds of silt and may form a considerable proportion of the silt deposits. The distribution of ice beds in many places is quite irregular and depends on conditions that are not understood. According to both Tyrrell (*18*) and Maddren (*19*), however, the ice originates from percolating water or from water under hydraulic pressure.

W. C. Buetow, State highway engineer, Wisconsin State Highway Commission (*20*), states:

> There are two kinds of boils, at least, in Wisconsin. The first appears—note I am not positive in this statement—to be caused by a live vein of water just under the subgrade. It usually appears on side hills. Troubles of this kind have in many instances been traced to a lake or swamp area which may be close at hand or miles away. The second type of boil, the kind that makes the most serious trouble, generally appears on the more level stretches of road and where the roadbed is built of a fine, medium clay, superimposed on a nonporous subsoil of exceptionally heavy clay or hardpan. The two types just described are the Wisconsin brand of boils. There may be others, but we are not familiar with them.

Differential frost heaving also may occur in deposits of silt or porous clay due to water raised by capillary tension from the ground water below. From what depths silts or clays at given densities may raise water in quantities harmful to the subgrade is not definitely known. Upon freezing, a stratum of soil ceases to receive water, but upon thawing a subsequent freeze may show a water content in excess of that found during the previous freeze. In contrast a wet frozen stratum of soil in contact with water beneath may become quite dry at the surface under conditions favorable to evaporation.

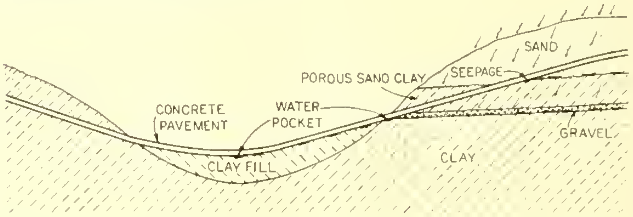


Figure 25.—Illustration of a Road Profile Containing Water Pockets Which May Produce Frost Heaves During the Winter

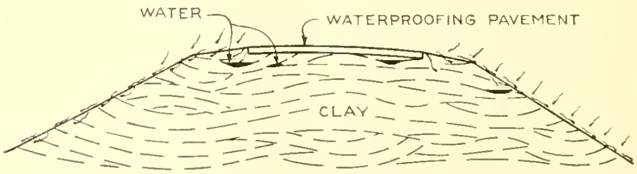


Figure 26.—Illustration of Water Pockets Due to Irregular Capacity of a Fill for Retaining Pore Water

CONCLUSIONS

It is apparent that the forces developed by action of freezing temperatures on water are enormous and may prove highly detrimental to highways and other engineering structures. From the foregoing discussion it will be seen that in order to minimize the destructive

(Continued on page 79)

PROGRESS REPORT ON THE CONNECTICUT AVENUE EXPERIMENTAL ROAD

MAINTENANCE AND BEHAVIOR OF SECTIONS DURING 1928 AND 1929

Reported by **PAUL F. CRITZ**, Associate Highway Engineer, and **J. H. ELDRIDGE**, Superintendent of Road Construction, Division of Tests, United States Bureau of Public Roads

THE Connecticut Avenue experimental road extending from Chevy Chase Circle to Chevy Chase Lake in Montgomery County, Md., was built during the years 1911, 1912, and 1913. The history of this project from the time of construction to 1928 is given in Public Roads, volume 9, No. 3, May, 1928.[1]

This report covers the maintenance and behavior of the various sections during 1928 and 1929. Figure 1 shows the location of the various sections, and the construction and maintenance costs are given in Table 1. Accumulated maintenance costs and traffic are shown in Figure 2. Analyses and quantities of materials used in construction are given in Tables 2 to 7, inclusive. The cost of past surface retreatments is included in Table 1, but the amounts of materials used have been

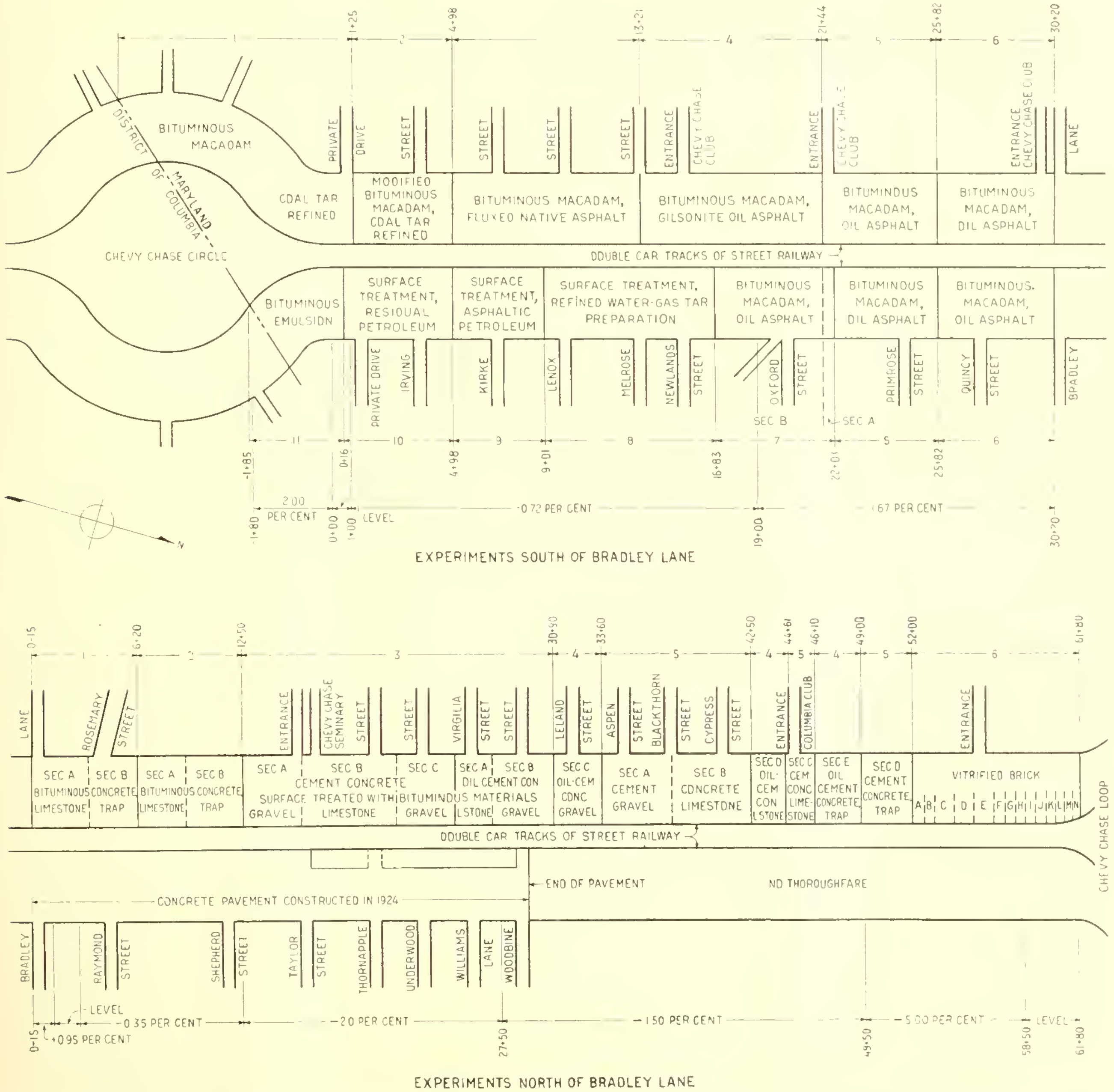


FIGURE 1.—LOCATION OF VARIOUS EXPERIMENTAL SECTIONS. THE GRADES ARE APPROXIMATELY THE SAME ON BOTH SIDES OF THE TRACKS AND THOSE DESCENDING TOWARD THE NORTH ARE SHOWN AS NEGATIVE

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[1] Reports describing the construction and early behavior of these experiments are included in Circulars 98 and 99, Office of Public Roads; U. S. Department of Agriculture Bulletins 105, 257, 407, and 586, and Office of the Secretary Circular 77.

TABLE 1.—*Cost and description of experiments on Connecticut Avenue, Chevy Chase, Md.*

BITUMINOUS MACADAM (PENETRATION) EXPERIMENTS, SOUTH OF BRADLEY LANE, BUILT IN 1911

Original construction						Annual cost of surface treatments and maintenance in cents per square yard											
						1912		1913		1914		1915		1916		1917	
Experiment No.	Length [1]	Area [1]	Bituminous material or type	Gallons per square yard	Cost of surface, per square yard	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance
	Feet	*Square yards*			*Cents*												
1	513	1,581	Coal tar, refined	2.59	46.18					5.04	0.74		2.34		0.28		0.29
2	373	705	Coal tar, refined (modified)	4.15	64.42								2.29		.64		1.49
3	823	1,555	Fluxed native asphalt	1.96	64.69								.95		.38		.11
4	823	1,555	Oil asphalt (Gilsonite)	2.19	57.18				0.26		.16		2.58		.67		.78
5	819	1,447	Oil asphalt	2.25	58.27		17.98		.15		.06		2.91		.53		.70
6	876	1,555	Oil asphalt	2.09	68.22				.22				2.07		.57		.18
7A	449	849	Oil asphalt	1.86	54.80	8.01					.15		2.89		.29		.45
7B	69	130	Oil asphalt	2.26	59.96		1.09				.34		3.59				
SURFACE TREATMENT EXPERIMENTS ON WATERBOUND MACADAM, SOUTH OF BRADLEY LANE, BUILT IN 1911																	
8	782	1,477	Water-gas tar preparation	0.54	[2] 39.59			5.46					3.40	6.55	1.37	5.59	0.37
9	403	761	Asphaltic petroleum	.53	[2] 42.01			5.21		4.71	1.18	7.50	.88	8.76	1.48	5.93	.71
10	482	1,013	Residual petroleum	.79	[2] 44.31				1.88	8.11	20.17	7.80	5.24	8.90	4.08	5.36	1.39
11	201	377	Native asphalt emulsion	3.32	[2] 81.51										1.56		1.35
EXPERIMENTS NORTH OF BRADLEY LANE, BUILT IN 1912																	
1	635	1,498	Bituminous concrete (Topeka specification) 2 inches thick on 6-inch 1 : 3 : 7 cement concrete		186.62								0.22				
2	630	1,400	Bituminous concrete (District of Columbia specifications) 2 inches thick, on 6-inch 1 : 3 : 7 cement concrete and seal coat of 0.51 gallon fluxed native asphalt		195.65						0.29		.26		0.25		
3	1,840	4,178	Cement and oil-cement concrete as in experiment No. 2, surface treated with various types of bituminous materials		154.95						.03		.06	7.81	.29		0.04
4	771	1,744	Oil-cement concrete, 1 : 1¾ : 3 and 5 pints residual petroleum per bag of cement		150.25						.12		1.05		1.02		.15
5	1,339	3,013	Cement concrete, 1 : 1¾ : 3		142.29						.07		.68		.28		
6	980	2,055	Vitrified brick, with base as in experiments Nos. 1 and 2, 2-inch sand cushion grouted with 1 : 1 sand-cement		258.21						.06		.02		.08		.06

[1] Some of the sections were of varying widths.

[2] Includes cost of wearing course.

TABLE 2.—*Analyses of bituminous materials used in original construction of the experiments south of Bradley Lane*

Experiment number	1 and 2	2	3	4	5	6	7	8	9	10	10	11
Material	Coal tar, refined	Coal tar refined (light)	Fluxed native asphalt	Gilsonite oil asphalt	Oil asphalt	Oil asphalt	Oil asphalt	Refined water-gas tar preparation	Asphaltic petroleum [1]	Residual petroleum [2] 1912	Asphaltic petroleum 1914	Native asphaltic emulsion
Specific gravity 25°/25° C	1.258	1.219	1.058	0.974	0.999	0.989	0.973	1.113	0.949	0.976	0.964	1.038
Specific viscosity, Engler:												
1. 100° C., 100 c. c										13.1		
2. 50° C., 50 c. c								15.5				
3. 25° C., 50 c. c									118		113.3	
Float test:												
1. 50° C., seconds	150											
2. 32° C., seconds		47								205		
Flash point, °C									37		40	
Burning point, °C									68		85	
Melting point, °C			46	52	74	47	90					
Penetration, 25° C., 100 g., 5 seconds			128	146	73	94	55					
Per cent loss, 163° C., 5 hours, 20 g			2.14	.87	.05	.68	.44		27.10	.13	26.46	[3] 3.35
Per cent loss, 105° C., 5 hours, 20 g									16.71			52.54
Penetration on residue			[4] 56	[5] 99	[6] 65	[4] 79	[7] 50					[8] 195
Float test on residue, 50° C., seconds									[9] 100		81	[9] 72
Float test on residue, 32° C., seconds									[8] 32	229	226	
Percentage soluble in CS_2			94.51	99.81	99.46	99.59	99.82		99.88	99.74	99.92	36.30
Percentage organic insoluble			1.00	.12	.50	.28	.15		.08	.23	.06	1.10
Percentage inorganic insoluble			4.49	.07	.04	.13	.03		.04	.03	.02	3.30
Bitumen insoluble in 86° B. naphtha			20.78	21.13	24.68	20.10	26.20		9.10	9.93	7.43	
Fixed carbon, per cent			10.64	7.79	13.53	8.36	10.15		4.91	7.67	5.04	
Free carbon, per cent	29.60	23.35						3.04				
Water												49.50
Ammonia												.35
Fatty and resin acids												9.45
Distillation, percentage by weight:												
Water	0	1.0						0				
Up to 110° C	[10] .2	[11] .7						0				
110° to 170° C	[10] .6	[12] .5						[11] .6				
170° to 270° C	[13] 10.2	[14] 16.8						[11] 24.8				
270° to 315° C	[10] 7.5	[15] 6.5						[11] 9.7				
Residue	[16] 81.4	[16] 74.3						[17] 64.9				
Total	99.9	99.8						100.0				

[1] Fairly thin fluid with strong naphtha odor.
[2] Viscous, sticky fluid.
[3] Loss in addition to loss at 105° C.
[4] Sticky, glossy surface.
[5] Mottled surface.
[6] Sticky, slightly mottled surface.
[7] Hard, fairly lustrous.
[8] Residue from percentage loss test at 105° C.
[9] Residue from percentage loss test at 163° C.
[10] Solid.
[11] Clear.
[12] Turbid.
[13] One-third solid.
[14] Two-thirds solid.
[15] One-sixth solid.
[16] Hard, dull, brittle.
[17] Sticky, semisolid. A 350° to 375° C. fraction showed 7.5 per cent insoluble in dimethyl sulphate.

TABLE 1.—*Cost and description of experiments on Connecticut Avenue, Chevy Chase, Md.*—Continued

BITUMINOUS MACADAM (PENETRATION) EXPERIMENTS, SOUTH OF BRADLEY LANE, BUILT IN 1911

Annual cost of surface treatments and maintenance in cents per square yard—Continued

Experiment No.	1918		1919		1920		1921		1922		1923		1924		1925		1926		1927		1928		1929		Total maintenance 1912 to 1929 inclusive
	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	*Cents per sq. yd*
1	19.12	1.47						0.04					9.68	1.37				1.90		0.72		3.43		0.26	46.38
2	22.42	2.95						.10						2.59						.89		11,51		2.52	47.40
3	20.84	1.40						.04						1.37				3.39		.65		1.04		1.25	31.42
4	19.40	1.63						.82						.99		8.96		3.51		3.29		13.02		0.00	56.07
5	16.61	2.75		0.32				.05						3.38		6.33		4.82				5.67		18.54	80.80
6	17.54	1.27						.04				1.51		3.77		1.34		2.37				5.77		1.08	37.73
7	20.93	1.98						.07				1.32		6.22		4.05		8.84		3.36		3.97		3.03	64.76

SURFACE TREATMENT EXPERIMENTS ON WATERBOUND MACADAM, SOUTH OF BRADLEY LANE, BUILT IN 1911

Experiment No.	1918		1919		1920		1921		1922		1923		1924		1925		1926		1927		1928		1929		Total maintenance 1912 to 1929 inclusive
	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	*Cents per sq. yd*
8		1.31	11.96	0.57				0.05				6.21	9.55	3.61		5.79	11.41	11.94		0.33		4.39		9.56	99.42
9	11.56	3.43	11.36	1.08	8.18		17.21	4.23				7.99		20.76		22.47		18.47		.65		25.15		30.39	219.28
10	9.11	2.04	10.49	.81	6.89	2.27	17.54	1.06				2.95		8.85		6.74		21.36		2.33		16.18		18.25	189.80
11	12.80	6.89			14.99	4.75		6.44				2.65	7.91	2.75		6.63		9.18		1.32		8.91		0.00	88.13

EXPERIMENTS NORTH OF BRADLEY LANE, BUILT IN 1912

Experiment No.	1918		1919		1920		1921		1922		1923		1924		1925		1926		1927		1928		1929		Total maintenance 1912 to 1929 inclusive
	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	Treatment	Maintenance	*Cents per sq. yd*
1				0.29				0.61		2.67		1.00		1.68		0.56		1.33		1.18		6.71		4.90	21.15
2				.30				.65		2.47		.79		1.93		1.06		.39		1.86		1.58		0.78	12.61
3		0.07		.20		0.39		.16	10.45	1.38		3.52		1.54		1.74		7.58		2.53		5.60		5.14	48.53
4		.29		.47		.94		1.93	10.46	3.31		4.23		4.99		1.68		11.90		4.86		3.26		4.90	55.56
5		.47		.27		.55		.22	10.45	1.69		3.49		1.55		.42		1.53		1.07		3.99		3.59	30.32
6		.09		.66		.66		.03		.66		.55				.28		4.42		1.69		[3] 1.28		[3] 2.11	12.65

[3] Cost of maintaining sections not affected by fill settlement.

TABLE 3.—*Character and extent of experimental sections constructed on Connecticut Avenue north of Bradley Lane*

Experiment		Location		Area	Type	Aggregate
No.	Section	From—	To—			
				Square yards		
1	A	0−15	3+19	1,498	2 inches bituminous concrete (Topeka specification).	Limestone.[1]
	B	3+19	6+20		----do	Trap.[1]
2	A	6+20	9+04	1,400	2 inches bituminous concrete (District of Columbia specification)	Limestone.[1]
	B	9+04	12+50		----do	Trap.[1]
3	A	12+50	15+84	4,178	Cement concrete surface treated with bituminous material.	Gravel.
	B	15+84	21+60		----do	Limestone.
	C	21+60	25+03		----do	Gravel.
	A	25+03	27+29		Oil-cement concrete, surface treated with bituminous material.	Limestone.
	B	27+29	30+90		----do	Gravel.
4	C	30+90	33+60	1,744	Oil-cement concrete	Do.
	D	42+50	44+61		----do	Limestone
	E	46+10	49+00		----do	Trap.
5	A	33+60	37+85	3,013	Cement concrete	Gravel.
	B	37+85	42+50		----do	Limestone.
	C	44+61	46+10		----do	Do.*
	D	49+00	52+00		----do	Trap.
6		52+00	61+80	2,055	Vitrified brick	

[1] Aggregate used in the bituminous concrete. Gravel used in the cement concrete of the base.

omitted from this report as no retreatments have been applied since 1926.

TABLE 4.—*Analyses of bituminous concrete mixtures*

Coarse aggregate	Experiment No. 1		Experiment No. 2	
	Topeka specification		District of Columbia specification	
	Limestone	Trap	Limestone	Trap
Bitumen soluble in CS_2	7.1	8.7	6.7	6.7
Sieve analysis of aggregate:				
Pass 1¼-inch screen, retained 1-inch screen			2.1	0
Pass 1-inch screen, retained ¾-inch screen			13.2	6.5
Pass ¾-inch screen, retained ½-inch screen	1.8	2.0	16.7	15.2
Pass ½-inch screen, retained ¼-inch screen	11.3	14.0	13.5	19.2
Pass ¼-inch screen, retained ⅛-inch screen	20.5	13.0	12.7	13.5
Pass ⅛-inch screen, retained 10-mesh sieve	14.5	8.8	6.5	7.4
Pass 10-mesh sieve, retained 20-mesh sieve	15.8	17.0	7.6	8.7
Pass 20-mesh sieve, retained 30-mesh sieve	5.8	7.3	3.5	4.0
Pass 30-mesh sieve, retained 40-mesh sieve	3.8	4.0	3.3	3.3
Pass 40-mesh sieve, retained 50-mesh sieve	1.6	2.3	2.0	1.9
Pass 50-mesh sieve, retained 80-mesh sieve	3.3	4.4	4.1	4.0
Pass 80-mesh sieve, retained 100-mesh sieve	1.5	1.8	1.1	1.2
Pass 100-mesh sieve, retained 200-mesh sieve	2.9	4.6	2.0	2.6
Pass 200-mesh sieve	10.1	12.1	5.0	5.8
Total	100.0	100.0	100.0	100.0

SECTIONS 1 TO 8 OF BITUMINOUS MACADAM CONTINUE TO GIVE GOOD SERVICE

Section 1, although still maintained by the bureau, has lost considerable of its value as an experiment due to the reconstruction work done by the municipal forces around Chevy Chase Circle in November and December

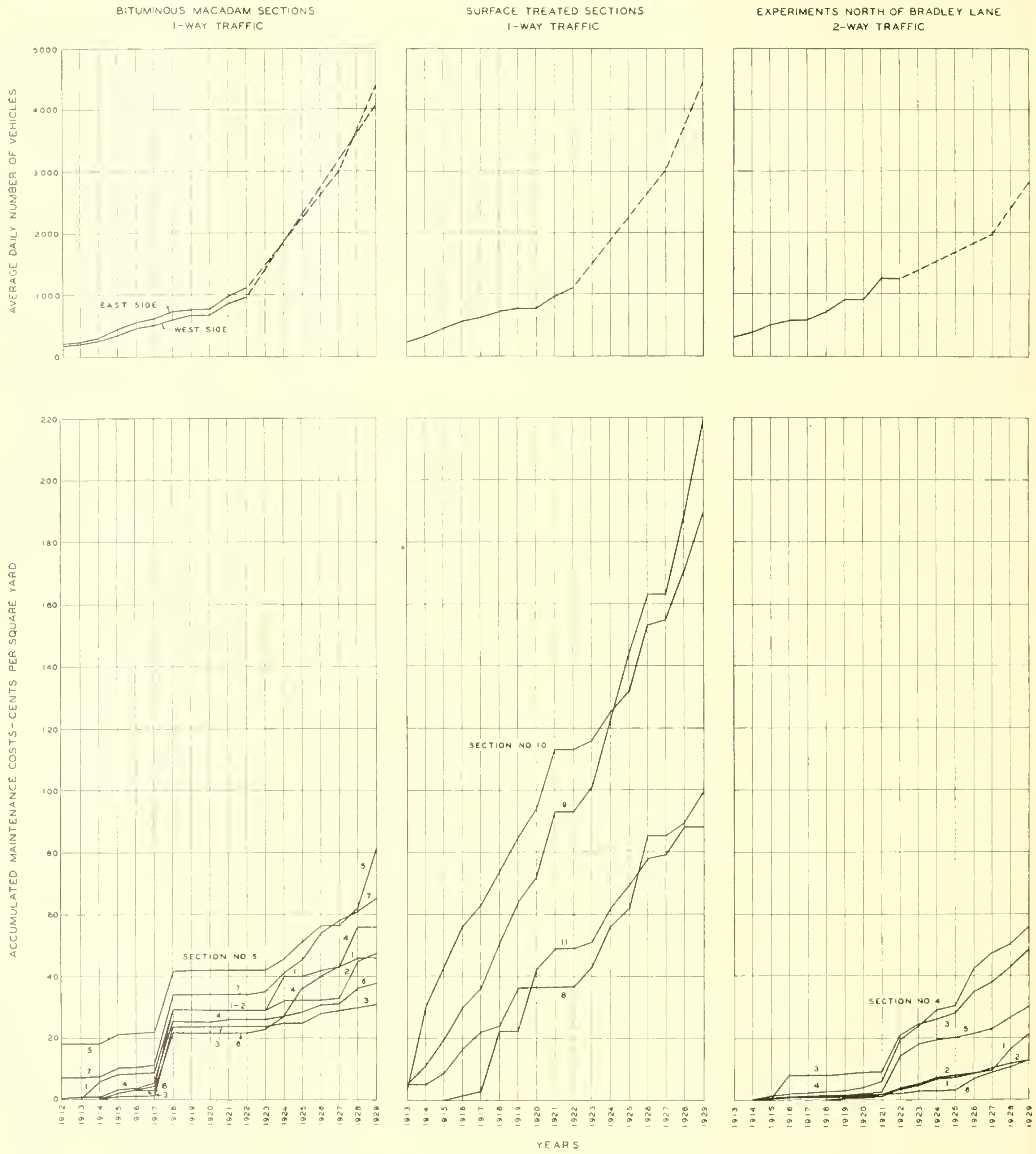

Figure 2.—Maintenance Costs and Traffic on Experimental Sections

1927. This work was incidental to relaying the curb and gutter and involved the rebuilding of a large portion of the section.

As shown in Table 1, the maintenance cost during 1928 for sections 1, 3, 5, 6, and 7 was about normal while that of sections 2 and 4 was somewhat higher than during former years. The maintenance on the latter two sections has consisted largely of repairing depressions which developed from foundation settlement, particularly along the west gutter. As stated in the previous report, the drainage afforded the sections south of Bradley Lane at the best was only fair. The street-car tracks offered a means for water to enter the foundations and the cobble gutters did not long remain satisfactory as drainage structures. The relatively high maintenance cost of section 5 in 1929 was also due to surface settlement. This section, in its early life, required considerable patching because of foundation

TABLE 5.—*Analyses of bituminous materials used in original construction of experiments north of Bradley Lane*

	Experiments Nos. 1 and 2: Bituminous concrete—fluxed native asphalt	Experiment No. 3 (surface treatments)							Experiments Nos. 3 and 4: Oil-cement concrete—residual petroleum
		Sections A and G: Refined coal tar	Sections D and F: Water-gas tar preparation	Sections B and H: Water-gas tar preparation	Section E: Fluxed native asphalt	Sections C, D and I: Fluxed native asphalt	Section F: Oil asphalt	Section J: Oil asphalt	
Specific gravity, 25°/25° C	1.074	1.219	1.108	1.144	1.045	1.043	1.031	1.012	0.933
Specific viscosity, Engler:									
1. 50°/50° C., 100 c. c									27.8
2. 30°/30° C., 50 c. c			14.0						
Penetration, 25° C., 100 g., 5 seconds	60				184	148	[1] 26	147	
Float test, 32° C., secouds		83		207					
Float test, 50° C., seconds		40		75					
Melting point, °C	53				39	48	92	52	
Loss 163° C., 5 bours, 20 g., per cent	2.86				3.50	2.73	.32	.59	2.63
Penetration on residue	29				63	64	18	103	
Float test on residue at 32 ° C., seconds									95
Per cent soluble in CS_2	93.56				94.70	96.56	99.74	99.72	99.90
Per cent insoluble in CS_2	1.86				1.23	1.35	.17	.17	.08
Per cent inorganic insoluble	4.58				4.07	2.09	.09	.11	.02
Per cent bitumen insoluble in 86° B., naphtha	23.83				19.60	21.87	34.84	21.72	2.31
Per cent fixed carbon	11.20				9.83	11.17	15.62	10.92	3.01
Per cent free carbon		16.29	.25	.95					
Distillation, per cent by weight:									
Water		0	0	0					
Up to 110° C		[2] .5	[3] 1.4	[3] .1					
110° to 170° C		[4] .2	[3] .4	[3] .2					
170° to 270° C		[5] 14.7	[3] 16.3	[3] 6.4					
270° to 315° C		[6] 8.7	[7] 17.0	[8] 10.4					
Residue		75.8	64.8	82.8					
Total		99.9	99.9	99.9					

[1] Penetration at 0° C. (200 g., 1 minute) 14; penetration at 46° C. (50 g., 5 seconds) 58.

[2] One-half solid.

[3] Clear.

[4] Cloudy.

[5] Two-thirds solid.

[6] Solid.

[7] Clear. Showed 7.5 per cent insoluble in dimethyl suplhate. A 315° to 350° C. fraction showed 7.5 per cent, and a 350° to 375° C. fraction showed 17.5 per ceut insoluble in dimethyl sulphate.

[8] Clear. This fraction and also a 315° to 350° C. fraction and a 350° to 375° C. fraction each showed 7.5 per cent insoluble in dimethyl sulphate.

weakness, and as the weak areas were eliminated the maintenance cost decreased. A renewal of this type of failure such as developed during the past year will undoubtedly affect the future behavior of this section, particularly in view of the larger volume of traffic now carried.

At the present time all of the sections are in good condition. The surfaces are somewhat wavy in spots but are intact and free from raveling. Except for possible failure due to foundation conditions, they should continue to give satisfactory service for some time at moderate cost.

TABLE 6.—*Mechanical analyses of coarse aggregates used in concrete Experiments 3, 4, and 5, north of Bradley Lane*

Size	Gravel	Limestone	Trap
Pass 2½-inch, retained on 1½-inch screen	2.3		
Pass, 1½-inch, retained on 1¼-inch screen	10.8		
Pass 1¼-inch, retained on 1-inch screen	24.0	13.2	6.6
Pass 1-inch, retained on ¾-inch screen	25.8		19.5
Pass ¾-inch, retained on ½-inch screen	26.4	34.2	40.6
Pass ½-inch, retained on ¼-inch screen	9.8	47.4	24.9
Pass ¼-inch	.9	5.2	8.4
Total	100.0	100.0	100.0

TABLE 7.—*Tests on vitrified brick used in experiment No. 6 north of Bradley Lane*

[Length of section, 973.1 feet]

Section	Length	Type of brick	Rattler loss	Water absorption	Description
	Feet		*Per cent*	*Per cent*	
A	51.5	Shale, wire cut lug	21.12	1.39	Hard-burned brick having a good structure.
B	67.5	do	16.36	1.31	Medium hard-burned brick having a very good structure.
C	108.7	Shale, re-pressed	25.57	.88	Brick well vitrified; losses in rattler mainly due to chipping.
D	105.0	do	17.67	1.65	Brick molded from coarsely ground shale; had a fairly good structure and was hard burned.
E	111.4	do	22.04	1.10	Brick very hard burned; losses in rattler due to chipping.
F	69.4	do	18.80	1.81	Brick molded from coarsely ground clay; had a good structure.
G	60.5	do	27.92	2.29	Medium hard burned brick which wear evenly though excessively in the rattler test.
H	67.9	do	22.68	3.74	Medium hard burned brick made from finely ground clay and having a fairly good structure.
I	50.0	do	22.59	2.86	Medium hard brick made from coarsely ground clay and wearing down uniformly in the rattler.
J	61.3	Fire clay, re-pressed	19.11	1.56	Brick made from coarsely ground fire clay; had an excellent structure, free from laminations; not burued very hard.
K	54.7	do	37.68	2.38	Comparatively soft-burned brick made from coarsely grouud fire clay; wear in rattler excessive though uniform.
L	58.8	Shale, re-pressed	38.89	4.04	Comparatively soft-burned brick made from coarsely ground clay; wear in rattler excessive though uniform.
M	60.1	Fire clay, re-pressed	24.31	3.73	Fairly soft-burned brick made from medium finely ground clay; worn down evenly by rattler.
N	51.3	Fire clay, wire-cut lug	31.19	3.68	Losses in rattler due mainly to open laminations; brick burned hard.

RESULTS ON SURFACE-TREATED SECTIONS INDICATE DESIRABILITY OF USE OF PRIMING COAT IN CONSTRUCTION

The work previously referred to, which was done around Chevy Chase Circle, likewise affected section 11 of this group.

During 1928 sections 8, 9, and 10 required fairly heavy maintenance. On sections 9 and 10 the surface mat slipped upon the stone base and became very wavy This displacement was intensified during the winter months when chains were used on cars. Where the base became exposed raveling developed. Defective areas were repaired by removing the bituminous mat to the base stone, which was then painted with tar and a patch made with a mixture of tar and stone. About 20 per cent of the areas of sections 9 and 10 were repaired in this manner.

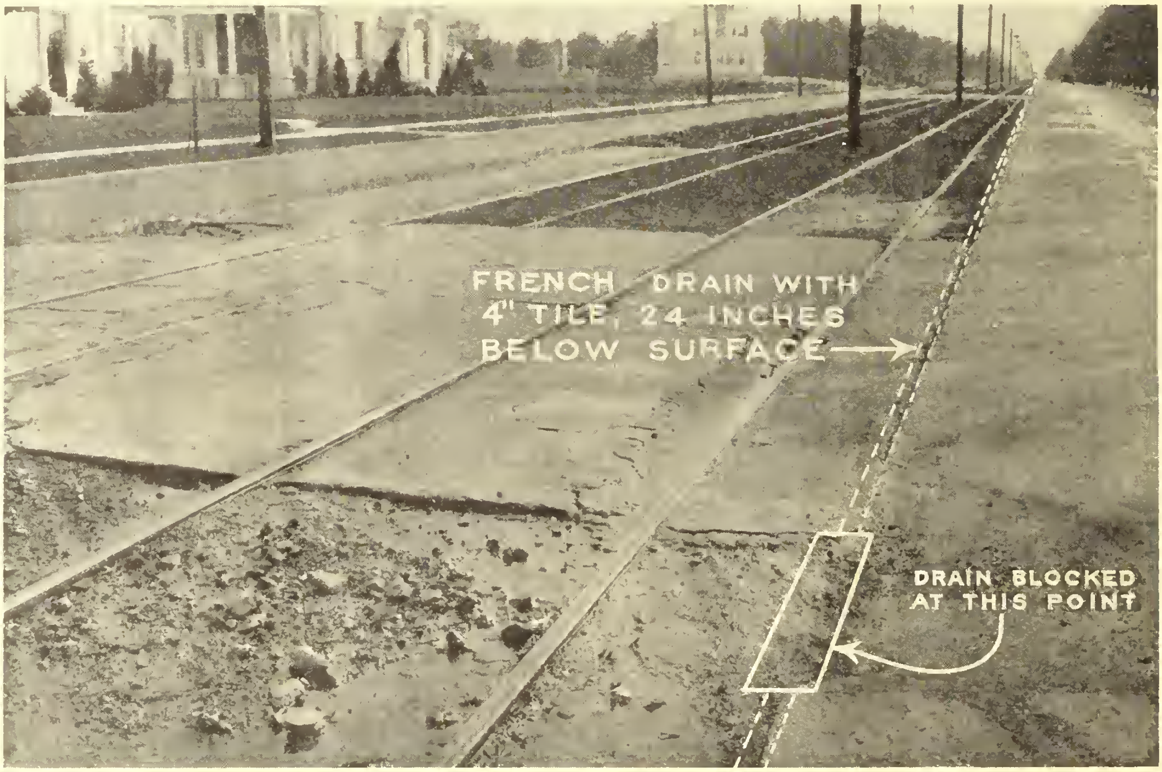
FRENCH DRAIN WITH 4" TILE, 24 INCHES BELOW SURFACE
DRAIN BLOCKED AT THIS POINT

Figure 3.—Section 3, North of Bradley Lane, February, 1929. Heavy Flow of Water Rising to the Surface Immediately Under West Rail on North Side of Virgilia Street Intersection. Note Bituminous Patch on the Right Which Was Necessitated by Settlement Caused by the Improper Back Filling of a Service Cut. The East Drain Was Stopped at this Point.

Maintenance for 1929 was normal except for sections 8 and 9. Section 8 because of its dry-appearing surface was practically covered with a light patch and at present is apparently in very good condition. Approximately two-thirds of the surface mat on section 9 had to be replaced. Except for a strip about 2 feet in width along the car tracks and one 3 feet wide next to the gutter, the surface is now uniform in appearance and the entire section is fairly smooth.

The behavior of the surface-treated sections during the past two years emphasizes still further the advantage of using a suitable priming material before applying a surface treatment. In a study of the Bradley Lane[2] experiments and those on the Department of Agriculture grounds[3] it was shown that it is practically impossible to retain stability in a bituminous surface mat unless the surface so treated is well bonded and of a character to which the treatment will adhere. It was observed that the surface treatments applied to a penetration macadam adhered satisfactorily and that a wear-resisting stable mat could be successfully built up. The same materials, however, applied directly to the unprimed, water-bound macadam proved unsuccessful as there was no bond between the treatment and the foundation.

Sections 9 and 10 have been repeatedly re-treated and have developed mats which are not well bonded to the base and which are consequently subject to displacement. On the other hand, section 8, on which a light water-gas tar was used, has always remained stable. The retreatments required during its early life were due to the fact that the light tar used would not hold the stone cover. This light tar penetrated readily and bonded the surface of the base but in so doing left no binder to hold the stone. For such a condition, subsequent re-treatments can be successfully used to build up a wear-resisting mat but, in the case of unprimed sections such as Nos. 9 and 10, additional treatments serve only to build up a thicker mat and add little to the durability of the structure as a whole. The surface mat merely rests upon an unbonded base which itself is subject to movement or displacement.

The cost of maintaining the surface-treated sections since the time of construction has been over three times that of the bituminous macadams, being 9.15 cents and 2.86 cents per square yard, respectively. Experiment No. 8 has remained the most economical of the surface-treated group but its cost of maintenance has been more than double that of the average of the bitu-

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[2] Reported in Public Roads, February, 1929.

[3] Public Roads, October, 1929.

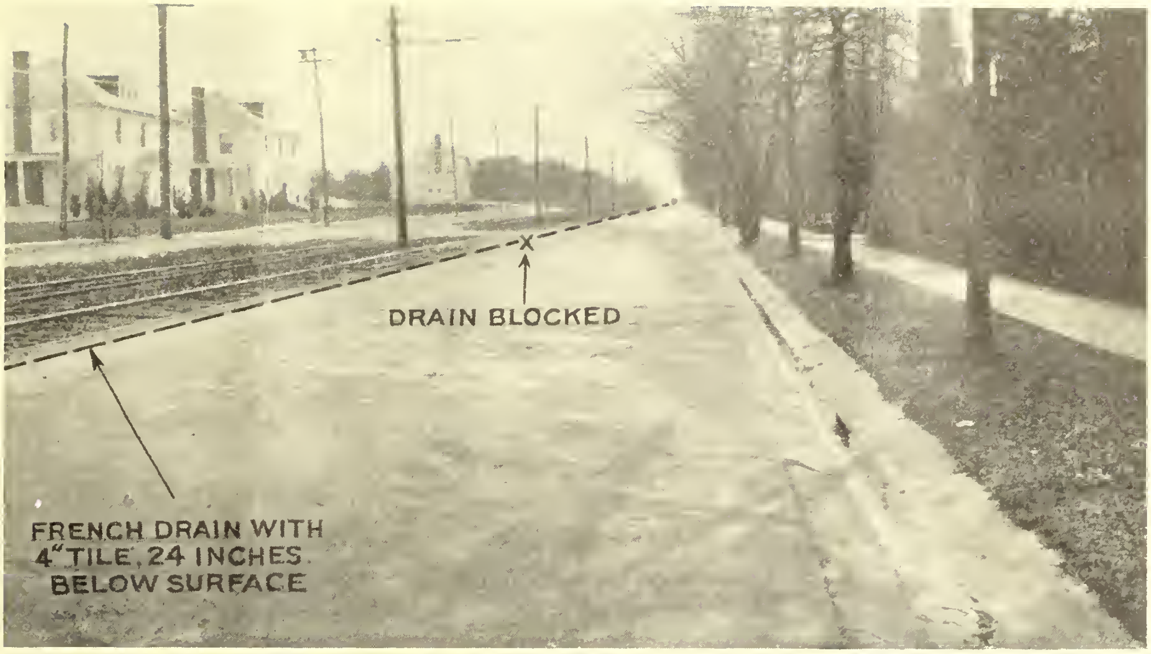

Figure 4.—Section 3, North of Bradley Lane, February, 1929. Appearance of Section Just North of Virgilia Street. Shiny Areas Are Water Which Has Come Up Through the Cracks in the Concrete Pavement. Note Water Which Has Flowed from the Surface Into the Gutter at the Right

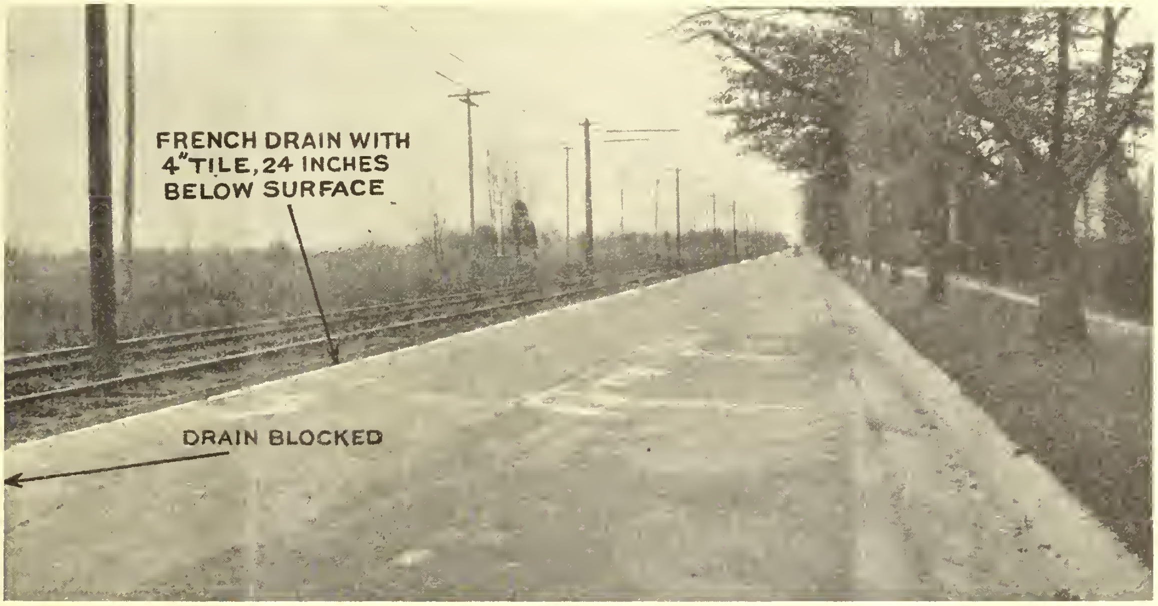

Figure 5.—View Taken Just North of Cypress Street and a Few Feet South of the Entrance to Columbia Country Club. Light Areas Are Pools of Water Collecting Along Cracks in the Concrete. Drain Line Was Blocked as Indicated

minous macadam sections. In this comparison of costs, section 11, which is more closely allied to penetration construction, has been omitted.

FAILURE OF DRAINAGE SYSTEM AFFECTS RESULTS ON SEVERAL SECTIONS

The six experimental sections north of Bradley Lane built in 1913 differ from the macadams and surface treated sections in that they are higher type surfaces laid upon a concrete base. Their construction and history have also been described in Public Roads of May, 1928.

As noted in that report, the subgrade upon which the six sections are located appeared such at the time of construction that French drains were deemed necessary on both edges of the pavement.

Maintenance during 1928 and 1929 consisted of routine repairs, practically all of which were caused by foundation failures.

An investigation of the drainage system made in February, 1929, by the bureau showed that it had practically ceased to function. Immediately following a heavy rain in February it was noted that a considerable volume of water was flowing to the surface of the concrete sections through the cracks. This condition was found on all the concrete sections but was more pronounced from Woodbine Street, south, which area embraces the whole of Experiment 3. It was also noted that the sewers were not receiving any flow from either side drain or through the cross drains. Upon digging down into the French drains it was found that wherever one of the utility companies had made a cut through the drain line they had failed to replace either the tile or stone but had back-filled their own trench with earth and thereby effectively destroyed the drainage system. The number of tiles thus removed from the line varied from eight to as many as twenty.

Preceding the failure of the drainage system the east drain line apparently took care of the area occupied by the street-car tracks, but with its failure this unpaved area offered an additional opportunity for surface water to enter the foundation of the experiments. At the time of inspection, February, 1929, it was observed that no free water was in evidence at Virgilia Street south of the intersection in the car-track area, but immediately north of the intersection, in the west car track, a considerable stream of water was rising to the surface, some of which flowed over the rail and onto the concrete pavement. This condition is illustrated in Figure 3. At this intersection the east drain was plugged up and as the accumulated water could travel no farther through the French drain, it flowed to the surface. This observed action was made use of in locating other breaks in the drain lines. Water was forced into the line under pressure until a flow to the surface indicated an obstruction. When the drain lines were uncovered at these points they were invariably found to be obstructed. When the defects had been corrected there was an unbroken flow through the east drain to its outlet at the north end of Experiment 6, and also on the west side of the sections which emptied into the sewer manholes.

The number of breaks in the tile line which were found and repaired was as follows: Experiment 1, 2 breaks; Experiment 2, 2 breaks; Experiment 3, 6 breaks; Experiment 4, 1 break.

The time at which the drainage system failed to function or, if it gradually failed, just what share of the maintenance required on the sections, or apparent slab failures were due to this fact, can not be estimated. The records of the sanitary commission show that most of the service connections were made in 1922, and it is interesting to note that the cost of maintaining the concrete sections has been materially higher since that date. The maintenance records show an average annual cost prior to 1922 of only 0.29 cent per square yard, but for the period from 1922 to 1930 it has averaged 2.86 cents per square yard. It was shown in the previous report that there was little variation in the compressive strength of the concrete cores taken, regardless of the size of the slab from which they were taken, and observations made revealed no evidence of structurally unsound concrete other than the numerous cracks which resulted in many small slabs. This excessive cracking and the corresponding increase in maintenance cost may have been influenced to some extent by the increased volume of traffic as shown in Figure 2, but it seems likely that they were influenced to a greater degree by foundation failures resulting from the causes described above.

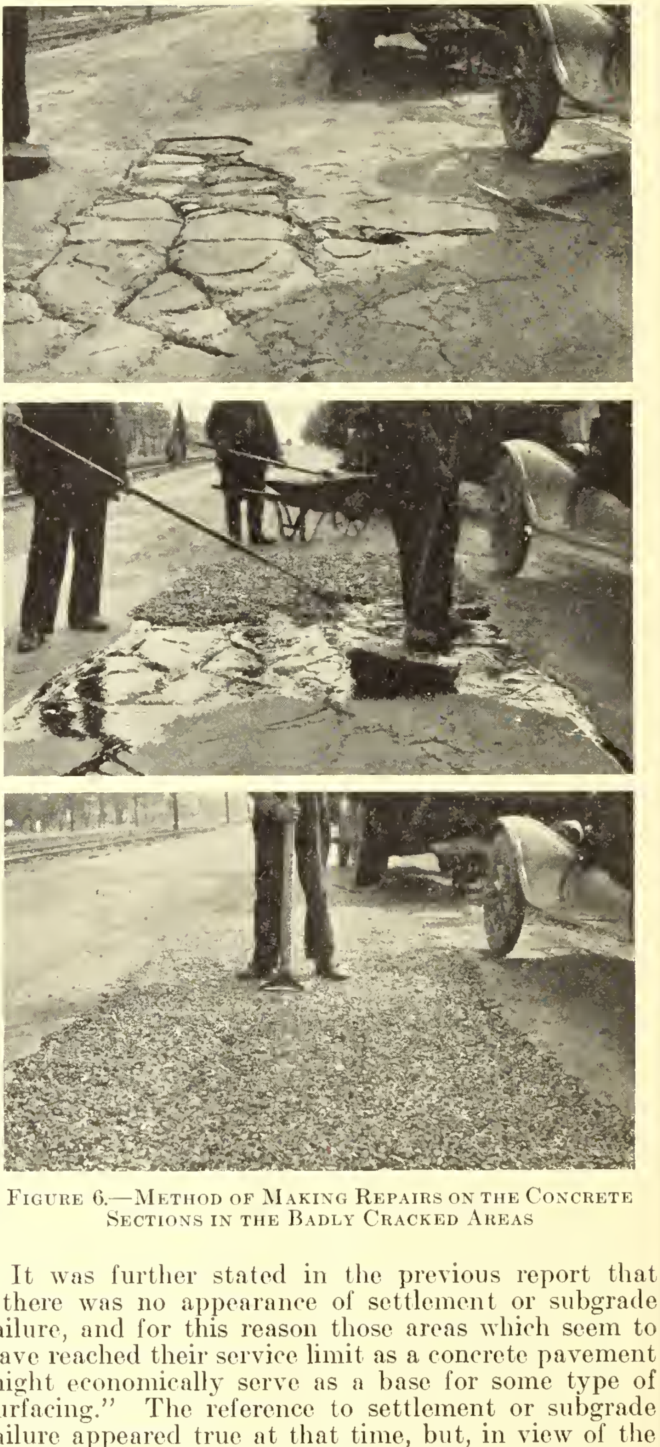

Figure 6.—Method of Making Repairs on the Concrete Sections in the Badly Cracked Areas

It was further stated in the previous report that "there was no appearance of settlement or subgrade failure, and for this reason those areas which seem to have reached their service limit as a concrete pavement might economically serve as a base for some type of surfacing." The reference to settlement or subgrade failure appeared true at that time, but, in view of the more recent study, the statement needs correcting. It is hardly reasonable to expect that such a great amount of cracking would have occurred had the slab received proper support. Just prior to February, 1929, some settlement of the concrete was noted, especially in the badly cracked areas. The amount was not great, but it did require the placing of many small patches to retain a reasonably smooth riding surface. After the drains were repaired and again began to function no slab settlement was noted and maintenance then consisted of filling cracks. There has been no recurrence of water coming to the surface, and, in general, the sections

(Continued on page 80)

SOME POINTS OF CONTACT BETWEEN SOIL SCIENCE AND HIGHWAY ENGINEERING[1]

By J. S. JOFFE, New Jersey Agricultural Experiment Station

THE principles underlying the elucidation of the natural sciences such as zoology and botany are applicable also to soil science. The soil is a natural body just as a tree or an animal. It is to be looked upon as a distinct organism with definite morphological and physiological features, with specific properties of physical build, chemical composition, and biological make-up. Soil science is concerned with the soil body as found in nature; its anatomy and physiology and its behavior toward the forces which are responsible for its creation. The first person to bring out the natural body features of the soil in relation to the forces in nature responsible for the creation of definite body types was a Russian scientist, Dokuchaev. The Russian school of soil science is known, therefore, as the Dokuchaev school.

Like any other of the natural sciences, soil science began with the descriptive phase. The soil body was dissected, cut open vertically, and the exposed anatomy noted and described. In a natural state it revealed a definite construction or build, consisting of distinct layers, known as horizons, which are specific in their morphological characters, irrespective of the geographic position of the soil or the underlying geologic formation provided it is located in identical climatic zones. The horizons exposed in a vertical cut of the soil body are genetically related and as a unit they represent what is known as the soil profile. Therefore a soil is a natural body consisting of definite layers or horizons made up of materials formed by a group of soil formers. Most of the materials that make up the soil body originate from the earth's mantle. The soil formers, which include the active factors such as the climate and biosphere, and the passive factors such as the parent material, micro-relief, age of land, and human activity, are responsible for the formation of the soil body.

It is to be understood that the parent material could be either native rock upon which the soil body has been formed or some material which might have been a part of a soil body before, as in the case of the so-called transported soils, or it might even be some geologic formation like peat, clay, marl, or sand. Soil material is therefore not to be identified with the soil as a natural body. A soil body ceases to be one as soon as its virgin make-up has been disturbed; it continues to be soil material from which a soil might form again in the course of time under the influence primarily of the active agents—the climate and the biosphere.

From what has been said it is clear that various parent materials will give rise to one and the same type of soil, provided all other conditions for the activities of the other soil formers are alike. And we do find the soil type known as chernozem on such variable parent material as loess, glacial deposits, marine and lake sands and clays, limestone, sandstone, and shale. On the other hand, on one parent material unlike soil types will develop, provided the conditions for the activities of the other soil formers are not alike. Thus granites in Georgia yield the typical yellow-red soil, whereas the same granites in southern California form a different kind of soil.

GEOGRAPHIC DISTRIBUTION OF SOIL TYPES DISCUSSED

Geographically, soils are distributed with a certain natural regularity in the same way as animals and plants. Just as any particular climatic belt is responsible for a definite flora and fauna, it is responsible also for a definite soil type or types. The habitus of the profile, its morphology, and chemical composition differ in each climatic belt. As a result, we have several zonal types of soil.

In Europe and Asia where the isohyetal lines are more or less parallel to the isothermal lines, i. e., as we move from the north southward the temperature increases, and the rainfall decreases. There the soil types are distributed parallel to the climatic belts; in the northern regions we find the tundra zone, a type of soil corresponding to this climatic zone: in the southern portion of the northern region and in part of the temperate region covered with conifers and deciduous forests, a type of soil knows as the podzol is distributed. South of this region—in the temperate region—the slightly podzolized type and forest steppe type of grayish brown soil is distributed, followed in the southern portion of the temperate region—where the somewhat semiarid regions are reached—with the chernozem soil. As one moves into the semiarid and arid regions, one finds the chestnut soils, and still farther south—in approaching the semidesert—one meets the gray soils. In the Tropics with a high temperature and high rainfall the well-known laterite type of soil is developed.

In the North American continent, especially in the eastern part, the direction of the isohyetal lines is, in general, perpendicular to the direction of the isothermal lines, and the geographic distribution of the zonal types does not follow the north and south direction as they do in Europe and Asia. It is these specific climatic features of the North American continent that necessitated a slightly different approach to the soils of the United States when studied from the standpoint of their profile development.

Dr. C. F. Marbut, the prominent American soil investigator, of the Bureau of Chemistry and Soils, United States Department of Agriculture, divided the soils of the United States into two large groups, (1) pedalfers,[2] and (2) pedocals.[2] The pedalfers are soils that tend to accumulate iron and aluminum and have no lime carbonate horizon accumulation, even if the soils have limestone as parent material. The accumulation of lime carbonate and other salts is a characteristic feature of the pedocals.

SOIL TYPES OF NEW JERSEY HAVE DEFINITE CHARACTERISTICS

For the present it will suffice to keep in mind these two broad groups. An analysis of the soil types found in New Jersey, which are located in the pedalfer group, will be in order. Those interested in the classification scheme in its entirety are referred to the paper by Doctor Marbut (*5*).

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[1] This paper is the summary of a lecture given before the Fifth Annual Short Course in Highway Engineering, Rutgers University. Journal Series paper, New Jersey Agricultural Experiment Station, Department of Soil Chemistry and Bacteriology.

[2] The prefix "ped" comes from the word "pedology"; "alfer" is apparently an abbreviation for aluminum and ferrum; the "cal" in pedocal comes from the word "calcium."

An examination of the virgin soils in New Jersey upon exposure of a profile cut will in general reveal the following: On the surface there is a dark-colored layer, 2 to 3 centimeters thick, of leaf mold consisting of (a) organic materials from the forest litter partly humified and partly in the process of being humified, and (b) some mineral soil material intermingled with the organic material mentioned and with the shallow roots of the herbaceous plants on the forest floor. This layer, or horizon, designated as the A_0 horizon is accumulative in character and is known as the humus-decay-accumulative horizon. Indeed, the leaves, twigs, and other residues from the trees, the bodies, and the roots of the herbaceous plants undergo decomposition and partly remain there, making up the volume of this horizon. It is also accumulative in another sense—the roots bring up some mineral substances from the horizons below; these are translocated to the plants, which give them up in the mineralization process of the organic matter. These mineral substances become partly fixed with the humus materials, and with the microbial flora which is instrumental in the decomposition of the organic matter, and they are partly leached downward. In this manner horizon A_0 is genetically related to the other horizons in the soil profile.

The horizon below, A_1, is usually yellowish brown in color, except in the soils that have red shale as the parent material; in this case the A_1 horizon is reddish brown. There is also some exception in color in the sandy soils in the southern part of the State in the Coastal Plain region. There the color of the A_1 horizon blends into a gray or white, and under such conditions this layer consists of bleached SiO_2, some of which is in very fine state of division, approaching at times the state of dust. In general, however, the yellowish-brown shade prevails.

This horizon is subject to the action of the decomposition products of the A_0 horizon, such as carbonic acid from respiration of roots and microbial decomposition of organic materials; nitric acid from the process of nitrification; some sulphuric acid from the process of oxidation of the sulphur compounds; and some organic acid from the decomposition of the organic materials. All of these acids leach this horizon and deprive it of its bases, its iron, and its aluminum, leaving behind some silica. The structure becomes less apparent; it becomes more powdery with fine porosity. With the downward movement of the moisture and the substances in it there is a tendency for some of the fine particles of clay also to move. The fine particles of organic materials are also caught in this downward movement.

This horizon is followed by another one, A_2, which is slightly lighter in color than the one over it. It also suffers the leaching reactions of the A_1 horizon. It gives up its bases, becomes acid, and loses some of its fine particles.

The lighter the texture of the parent material, the deeper are the horizons A_1 and A_2; and conversely, the heavier the texture, the shallower are these horizons.

Because of the losses which the *A* horizon (A_1 and A_2) suffers, it is known as the horizon of eluviation (washing out). Its depth varies from 15 to 60 centimeters. This horizon loses its bases and becomes enriched with silica. Some amorphous silica moves downward.

The materials which are leached out from the *A* horizon and are moved chemically and mechanically, are caught in the next horizon, which is known as the horizon of accumulation, or illuviation (washing in). It is designated as the *B* horizon. In it the fine clay particles, the iron and aluminum precipitates, and the finely divided humus materials make up a compact layer which at times becomes so cemented that it is impervious to water. Then it is known as ortstein formation, or hardpan. The depth of this horizon varies in the soils of New Jersey from 15 to 40 centimeters.

It is this layer which should primarily interest the highway engineer. Wherever an impervious layer *B* is formed—and it is in the more sandy soils that such a layer is likely to occur—this moisture condition, upon which the stability of the substance depends, is the most important consideration. Because of its fine texture this horizon has a tendency to attract the moisture from the layers below (the parent material) and above. The capillary moisture in the soil profile moves not only vertically but also horizontally and usually along this layer of accumulation.

It is to be remembered that this particular imperviousness exists in this zonal type of soil formation. In the pedocals there is an accumulation of calcium carbonate or sulphate, which serves as a flocculating agent, and hence the *B* horizon under such conditions is not impervious. The use of calcium carbonate or gypsum might be suggested in highway construction, whenever the subgrade consists of the *B* horizon which approaches a condition of orstein or hardpan. Such treatment might flocculate the colloids and make the horizon pervious.

In places where the water table is high, the *B* horizon becomes enriched with substances from the ground waters and as a result is sticky and mottled. A subgrade of this kind gives poor drainage. Whenever such a *B* horizon is encountered—and it may be recognized by its grayish and sometimes bluish color, occasionally with streaks of brown—it is a sure indication of a high-water table at some time during the year. Such an accumulation is known as gley. Its distinctive character is that iron compounds, because of poor aeration, are reduced. The state of oxidation of the iron compounds, as expressed by the coloration, is a true indication of the amount of gleying process this layer is subjected to. In other words, from the color one might infer the frequency and time duration of the ground waters coming up to this horizon.

Below the *B* horizon comes the parent material which is very little, if any, affected by the soil formers which go to make up the soil body with its characteristic profile. The parent material is usually designated by the letter *C*. Beyond a certain depth from the surface of the *C* horizon the soil scientist looks on the material as geologic formations, and the subgrade properties of the *C* horizon are to be elucidated more by geologic data than by soil data. The highway engineer should apply his constants to this material just the same as to any other material, be it sand, clay, or peat.

HIGHWAY ENGINEERS SHOULD MAKE USE OF SOIL PROFILES

There are a few other points which might be mentioned in connection with the possible aid a soil profile study could offer the highway engineer. In the well-known podzol zone type of soils there is a horizon of white coloration, without structure, fine porosity, sticky and smeary when wet, but powdery and almost dusty when dry. A layer of this kind has no stability and if one occurs close to the grade line it should be taken out and wasted. Such a layer is easily recognized by a trained soils man.

It is to be remembered that the border lines of the horizons in the soil profile, although very distinct, do not run parallel to one another. The microrelief of the land will to a certain degree undulate the contours; that is, the horizon will not be alike in thickness throughout. In fixing the grade line of the road, this condition should be considered. It is often possible to fix the grade line well above or below unstable material.

Speaking of the relation of the soil properties to highway engineering one could not fail to note the possible effect of shrinkage of peat materials when lime is added. Cases are known where a fill on peat has within 15 years subsided an amount equal to 15 to 25 per cent of its original depth. Although the engineer undoubtedly takes into consideration shrinking values—and in this respect his constants are in a way determined by the chemical and physical properties of the material, whether soil material or some other material—he does not consider the possibility of chemical or biological action after the road has been put down. The peat section coming in contact with the lime leached out from the roadbed might easily become active biologically, and shrinkage would take place, a consideration unforeseen by the highway engineer.

Heaving is one of the evils in which both the highway engineer and the soils man are interested. Heaving is controlled to a certain extent by the pore space. Heavier soils with a fine porosity heave the most and flocculating the clay increases the size of the pores aggregating the colloids. At this point it is well to remember the volume relations of soils when saturated with one cation or another. Thus a soil saturated with one cation will have a different volume than when saturated with another. The introduction of certain materials in the fill might influence the cation relations in the subgrade horizon.

In laying a grade line, the engineer under certain conditions of relief could follow the compact *B* horizon—of course having determined at first its adaptability as a subgrade—and in some cases where cuts are made just to get fill material it would perhaps be better not to touch the *B* layer and to borrow fill material from the adjoining land. The profile constitution survey along the projected road would tell the engineer a good deal in respect to the depth one should excavate for the best subgrade as it is found in nature.

In soils with sand as the parent material the profile is deep, hence the *B* horizon of accumulation is located deeply. For a sand-clay road where clay is essential as a binder one would have to dig to some depth in such a sandy soil to obtain the necessary clay binder. A knowledge of soils would indicate the presence of a *B* horizon with a higher clay content than at the surface.

In the field operations many other points of contact between the pedologist and the highway engineer will come up, and their cooperative efforts will undoubtedly benefit the science and the practices of highway engineering.

(Continued from page 68)

effects of frost action on highways, procedure must be adopted which will accomplish the following purposes:

1. Prevent free water from entering the subgrade either through the surface or through seepage veins.
2. Rapidly remove free water which may be liberated during thaws.
3. Where the subgrade is lower than the adjacent ground surface lower the ground water table to an extent which will prevent harmful frost action.
4. Where other methods would not be wholly effective, remove subgrade material possessing detrimental properties to the depth of frost penetration.

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(Continued from page 76)

at this time are in better condition than in January, 1929.

The behavior of section 3 especially, which has the worst appearance of any of the six experiments, makes it difficult to decide if this pavement has reached its service limit. The section as a whole is broken into a great many slabs of varying sizes and shapes, many of which are not more than 1 foot in area. In filling the cracks it is practically impossible to avoid leaving a slight ridge with a corresponding reduction in surface smoothness. However, to maintain as much smoothness as possible, some of the small areas have been entirely covered with a bituminous patch. This procedure is illustrated in Figure 6.

ASPHALTIC CONCRETE AND BRICK SECTIONS CONTINUE IN GOOD CONDITION

Maintenance of the two asphaltic concrete sections consisted of patching depressions, mostly along the west gutter line. Section 1, south of Rosemary Street, had become quite rough on the west side for a width of approximately 7 feet. It was first patched and then given a light treatment of cold tar and torpedo sand. These repairs were necessitated by the failure of the concrete base which, as stated in the previous report, had, in some areas, disintegrated to such an extent that it could be removed with shovels. This failure was more extensive on section 1 than on section 2. The surfaces of both sections are in very good condition at the present time.

The brick section, section 6, remains in excellent condition and shows little wear and no indication of failure due to the character of the surface. As shown in the preceding report, the concrete base of this experiment, excepting subsections K, L, M, and N, is in very good condition. Subsections K, L, M, and N lie on a fill which has settled and still continues to settle. Maintenance of these sections of the experiment has been high, but as it is not properly chargeable to the experiment it has been omitted from the table and curves and only the sections not affected by the fill are included.

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Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
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*370D. The Results of Physical Tests of Road-Building Rock. 15c.
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1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

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MISCELLANEOUS CIRCULARS

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Report of a Survey of Transportation on the State Highway System of Ohio.
Report of a Survey of Transportation on the State Highways of Vermont.
Report of a Survey of Transportation on the State Highways of New Hampshire.
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio.
Report of a Survey of Transportation on the State Highways of Pennsylvania.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

MAY 31,1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,140.5	$ 2,244,975.74	$ 1,109,342.18	70.6	22.1	92.7	$ 348,879.14	$ 172,648.16	25.9	0.1	26.0	$ 4,043,176.79	Alabama
Arizona	815.7	4,023,428.10	3,015,643.32	121.7	144.0	265.7	[illegible]	[illegible]	15.8	19.2	35.0	1,803,346.95	Arizona
Arkansas	1,736.4	5,518,596.25	2,571,866.18	140.2	33.5	173.7	1,041,160.03	418,357.47	56.4	21.4	76.8	2,131,678.14	Arkansas
California	1,913.9	6,723,559.19	2,666,091.28	123.5	30.1	153.6	1,120,592.67	589,515.70	39.3	8.0	47.3	2,508,229.83	California
Colorado	1,179.6	4,989,097.91	2,631,197.67	198.6	38.7	237.3	982,180.26	503,492.17	25.7	38.1	63.8	2,613,865.77	Colorado
Connecticut	240.4	2,271,691.32	981,371.99	10.7		10.7	608,889.83	163,215.00	10.9		10.9	778,000.92	Connecticut
Delaware	258.2	833,182.60	323,794.82	23.5		23.5	753,232.15	373,047.20	39.6		39.6	114,750.47	Delaware
Florida	503.5	4,924,091.30	2,257,258.51	97.1	5.5	102.6						1,782,889.95	Florida
Georgia	2,735.6	1,371,094.82	668,100.32	64.9		64.9	3,048,459.42	1,389,856.91	60.5	69.2	129.7	3,730,212.86	Georgia
Idaho	1,193.9	993,200.97	599,624.63	80.0	27.8	107.8	411,317.05	227,676.25	13.6	2.0	15.6	1,682,623.69	Idaho
Illinois	2,048.9	15,084,983.13	6,626,471.25	421.4		421.4	5,023,125.17	2,161,638.01	121.2	46.7	166.9	5,820,179.65	Illinois
Indiana	1,372.6	7,954,051.02	3,751,677.52	248.9		248.9	659,647.13	329,823.56	23.1		23.1	2,419,789.39	Indiana
Iowa	3,024.6	5,278,276.08	2,279,591.15	48.6	132.0	180.6	2,991,541.40	1,231,738.70	29.3	72.7	102.0	1,892.77	Iowa
Kansas	2,828.6	6,122,424.82	2,430,583.03	258.5	19.8	278.3	735,611.38	336,434.71	13.0	24.8	37.8	2,419,291.68	Kansas
Kentucky	1,452.1	3,855,007.56	1,821,064.33	197.2	4.2	201.4						2,814,563.49	Kentucky
Louisiana	1,355.7	4,563,418.03	2,235,056.64	142.9	12.5	155.4	705,953.08	335,603.31	20.9	4.9	25.8	1,892,079.68	Louisiana
Maine	521.5	1,828,390.73	714,287.56	46.0		46.0	1,911,044.89	642,651.39	46.9	1.6	48.5	1,382,176.03	Maine
Maryland	634.8	1,528,340.05	728,547.42	46.9	8.9	65.8	1,052,157.46	492,663.67	31.4	3.7	35.1	237,047.93	Maryland
Massachusetts	657.6	4,351,357.45	1,373,616.94	65.8	2.6	68.4	38,739.25	16,194.60	.2		.2	2,367,062.18	Massachusetts
Michigan	1,612.1	10,170,007.37	4,310,883.95	222.2	30.5	252.7	552,676.17	255,970.00	21.4		21.4	3,096,496.46	Michigan
Minnesota	3,976.0	10,196,442.72	3,586,778.31	240.6	207.9	448.5	1,823,354.82	722,175.40	11.1	61.8	72.9	24,716.29	Minnesota
Mississippi	1,810.5	1,846,209.28	716,929.45	65.6	7.7	73.3	48,835.05	24,417.52	.1		.1	3,527,874.48	Mississippi
Missouri	2,469.0	6,500,765.60	2,371,520.01	91.4	64.1	155.5	6,140,108.71	2,088,861.06	103.5	49.8	153.3	790,975.38	Missouri
Montana	1,705.5	7,870,639.46	4,591,486.48	528.2	29.7	667.9	1,398,987.89	799,242.42	113.8	50.7	164.5	3,505,060.50	Montana
Nebraska	3,639.5	7,378,837.40	3,484,632.27	314.2	133.1	447.3	1,471,193.48	584,268.63	46.8	75.0	121.8	2,476,362.21	Nebraska
Nevada	1,200.0	914,357.10	812,406.23	63.4	87.8	141.2	364,874.35	324,190.82		50.2	50.2	1,030,357.56	Nevada
New Hampshire	350.6	1,080,267.65	347,551.06	19.0	2.1	21.1						456,876.42	New Hampshire
New Jersey	504.8	6,087,677.22	1,520,866.32	69.7		69.7						977,173.76	New Jersey
New Mexico	1,886.4	4,105,339.92	2,687,028.09	203.7	50.6	254.3	319,209.24	296,765.07	16.1		16.1	1,141,623.34	New Mexico
New York	2,463.2	19,148,817.76	3,906,005.00	261.1		261.1	8,[illegible]85,750.00	1,617,600.00	108.0		108.0	7,930,830.36	New York
North Carolina	1,769.3	2,886,215.94	1,425,331.25	164.9	22.6	187.5	1,254,248.73	601,459.11	17.9	5.0	22.9	2,719,753.11	North Carolina
North Dakota	4,168.3	2,207,451.13	1,029,237.21	411.4	121.8	533.2	1,142,199.07	564,294.49	132.4	232.7	365.1	1,710,232.94	North Dakota
Ohio	2,180.7	15,477,502.95	4,846,972.65	276.2	18.8	295.0	8,523,045.84	2,824,946.56	145.4	31.2	176.6	1,965,113.48	Ohio
Oklahoma	1,894.3	3,204,617.69	1,400,538.15	95.3	40.1	135.4	2,667,096.43	1,190,362.64	71.9	42.5	114.4	913,420.10	Oklahoma
Oregon	1,172.7	3,550,316.69	2,130,614.53	168.3	61.9	230.2	2,176,969.99	1,255,088.02	61.6	23.4	85.0	554,857.14	Oregon
Pennsylvania	2,287.4	17,885,222.29	4,381,259.92	256.6	14.1	270.7	4,402,365.86	1,728,902.74	40.1		40.1	2,437,641.70	Pennsylvania
Rhode Island	184.8	1,827,490.55	546,833.18	27.8		27.8	129,559.02	64,779.50	.4		.4	645,410.42	Rhode Island
South Carolina	1,898.3	3,701,357.29	1,419,947.28	104.6	24.0	128.6	3,799,401.24	1,474,579.71	45.8	99.3	145.1	210,291.41	South Carolina
South Dakota	3,437.0	4,396,141.58	2,373,749.57	471.7	124.0	595.7	585,051.42	299,930.53	50.8	56.8	107.6	1,266,452.80	South Dakota
Tennessee	1,230.8	2,734,626.44	1,248,529.22	103.8	12.5	116.3	2,054,076.74	794,392.42	61.4	27.6	89.0	2,350,474.70	Tennessee
Texas	6,840.5	11,213,047.76	4,499,483.24	374.7	72.2	446.9	4,095,666.93	1,606,742.26	107.8	79.5	187.3	5,401,677.20	Texas
Utah	988.9	1,052,178.37	724,457.39	49.7		49.7	652,094.60	476,192.35	38.6	89.6	128.1	799,257.24	Utah
Vermont	255.2	2,111,448.16	746,478.72	37.4	2.5	39.9	409,314.86	125,441.63	7.4		7.4	12,427.09	Vermont
Virginia	1,427.3	4,832,336.44	2,285,539.06	198.4	25.5	223.9	891,195.68	437,571.24	33.4		33.4	642,406.39	Virginia
Washington	933.3	3,350,076.10	1,344,800.00	68.1	40.3	108.4	802,420.05	452,600.00	30.1	6.5	36.6	1,325,673.29	Washington
West Virginia	725.2	2,922,786.34	1,182,192.93	70.5	12.6	83.0	1,162,743.13	373,570.86	11.7	28.1	39.8	811,094.06	West Virginia
Wisconsin	2,244.6	5,261,613.82	2,100,385.13	158.2	22.4	180.6	2,858,720.65	1,210,275.00	60.8	31.5	92.3	1,197,921.57	Wisconsin
Wyoming	1,735.0	1,587,919.58	1,029,716.26	138.8	46.4	185.2	502,920.49	329,810.55	31.3	76.5	107.8	1,083,530.09	Wyoming
Hawaii	41.2	853,565.90	359,459.43	21.5		21.5						1,682,039.16	Hawaii
TOTALS	83,646.5	249,813,292.57	102,086,699.02	7,674.0	1,756.8	9,430.8	80,941,981.00	32,207,313.39	1,942.2	1,429.1	3,371.3	93,220,668.72	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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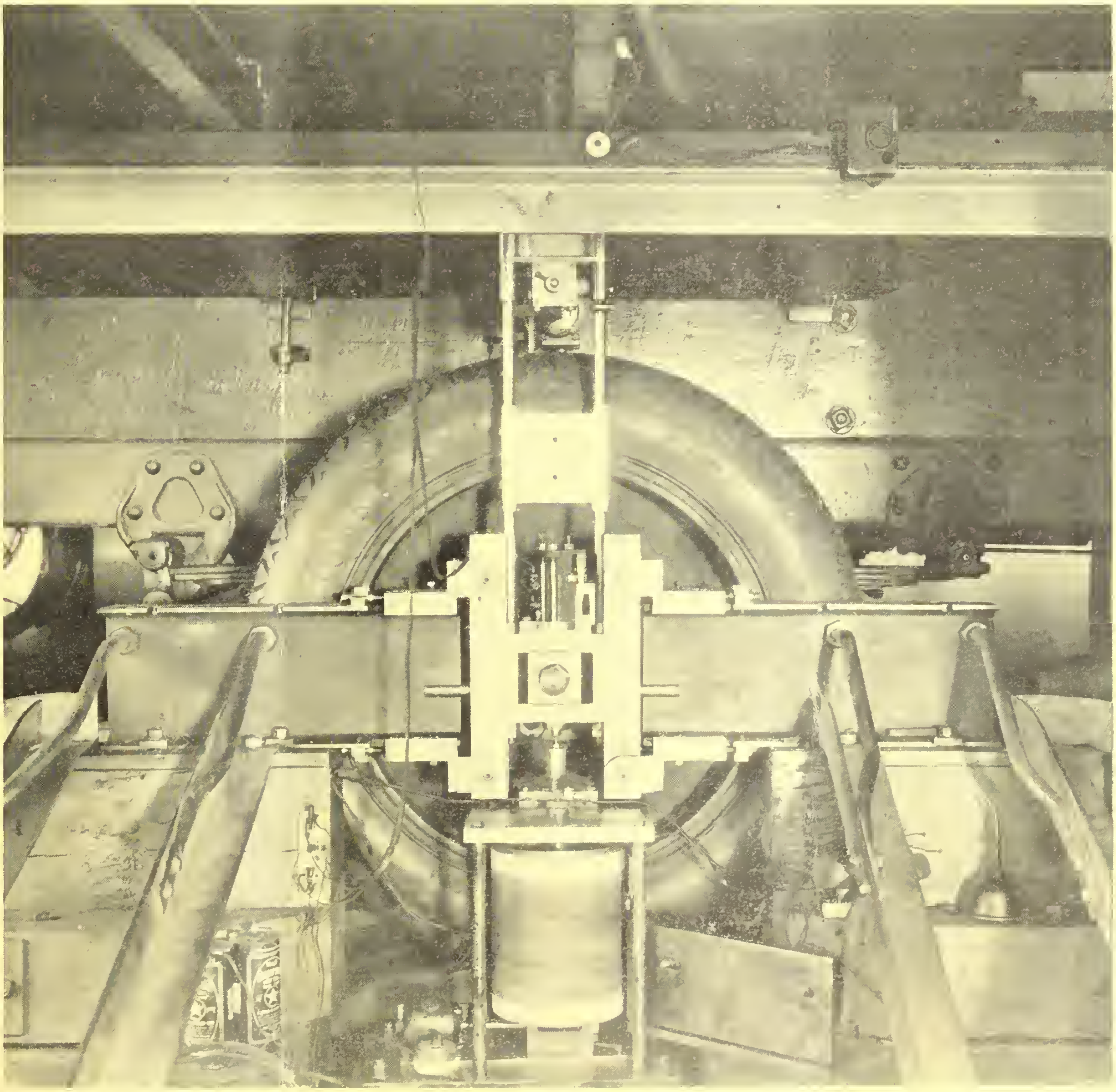
A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 5 ▽ JULY, 1930



LABORATORY APPARATUS FOR MEASURING MOTOR TRUCK IMPACT

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

Vol. 11, No. 5 JULY, 1930 G. P. St. CLAIR, Editor

TABLE OF CONTENTS

Page

Calibrations of Accelerometers for Use in Motor Truck Impact Tests 81

Program for International Road Congress Announced 110

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CALIBRATIONS OF ACCELEROMETERS FOR USE IN MOTOR TRUCK IMPACT TESTS

REPORT ON COOPERATIVE INVESTIGATION BY U. S. BUREAU OF PUBLIC ROADS AND U. S. BUREAU OF STANDARDS [1]

Reported by **J. A. BUCHANAN**, Associate Engineer of Tests, and **G. P. ST. CLAIR**, Associate Engineer of Tests, U. S. Bureau of Public Roads

This report deals with an investigation of three methods of measuring accelerations due to motor truck impact. They are, (1) by analysis of records from the displacement-time apparatus, (2) by means of the cantilever-spring, contact accelerometer, and (3) by means of the coil spring accelerometer. These instruments are described and the theory involved in the use of each is given. The calibrations included static tests, dynamic tests on a machine producing simple harmonic motion having periods and amplitudes analogous to those of motor truck impact, and dynamic tests with a motor truck on a reaction wheel closely simulating road-operating conditions. Data are given showing the agreement between basic measurements of acceleration, the relation between theory and practice in the case of the coil spring accelerometer, the dispersion to be expected in coil spring accelerometer data, and the estimation of impact periods. Comparisons are made which show the effect that new calibration data would have on impact forces computed by original calibrations. The report closes with a statement of conclusions reached as a result of the investigation, which may be briefly stated as follows:

1. The contact type of accelerometer may be used to obtain highly accurate measurements of acceleration of the order of magnitude and period encountered in motor truck impact work.
2. The coil spring accelerometer may be used to obtain reasonably accurate measurements of acceleration due to motor truck impact, when due consideration is given to the relation between the period of the impact reaction and the period of the accelerometer element.
3. A recomputation of published data using calibration factors obtained in this investigation indicates that the impact reaction values which are based on the original calibration factors are from 10 to 15 per cent too low, because of systematic errors in calibration. The comparisons made and conclusions drawn in reports based on such data, being general in nature and depending upon many tests rather than upon individual measurements, show with sufficient accuracy the relations and factors which influence the magnitude of the impact reactions.
4. The displacement-time apparatus may be used under laboratory conditions to determine the magnitude of the acceleration, the period of impact, and other characteristics of motor-truck impact reactions.

IN 1923 the Bureau of Public Roads, in cooperation with the Rubber Association of America and the Society of Automotive Engineers, commenced a series of tests to determine the magnitude of the impact reactions between a truck wheel and the pavement and to study the major factors influencing the magnitude of such reactions. The impact force was determined indirectly by computation, using the acceleration and mass of the unsprung weight on the truck wheel and the truck spring pressure or sprung weight at the instant of impact. To measure such accelerations an instrument known as the coil spring accelerometer was developed and used by the bureau.

This coil spring accelerometer was calibrated by comparing readings obtained from it with the magnitude of accelerations determined by the analysis of displacement-time records of the primary motion. A special impact testing machine was used to drop a truck wheel (to which the accelerometer was attached) on a pavement slab. It was found that, for use in tests involving any particular type of tire, a certain range in the characteristics of the sensitive element, i. e., spring-weight combination, of the accelerometer produced satisfactory results. A system of calibration factors was set up for each sensitive element for use with the tire equipments for which it was adapted.

REASONS FOR ACCELEROMETER INVESTIGATION OUTLINED

Prior to the publication of the first report on these impact tests, an investigation was made to determine the accuracy with which the accelerations of motor-truck impact were measured by the coil spring accelerometer. This investigation, which was also cooperatively conducted, resulted in the finding that the instrument was sufficiently accurate for use in procuring data from which only generalized comparisons were made. The first impact report showing the influences of various factors on the magnitude of motor-truck impact reactions was then released and appeared in PUBLIC ROADS, June, 1926.

Two additional research projects in motor-truck impact were planned to supply definite information concerning immediate problems. One was to determine the influence of the thickness of tread rubber (in solid and cushion types of tire) on the magnitude of impact reactions produced. The other was to establish the correlation between road roughness, tire equipment, wheel load, and vehicle speed in influencing the magnitude of impact reactions under actual operating conditions. While working on these projects, the Bureau of Public Roads, in the summer of 1926, prepared for an extended investigation of the coil spring accelerometer under well-controlled laboratory conditions which closely simulated the reaction between the truck wheel and the road.

The two motor-truck impact projects were completed in the fall of 1927, prior to the completion of the extensive instrument investigation. The reports on the impact tests were approved for publication, but before this was accomplished certain objections were made concerning the accuracy of the data. The bureau then decided to withhold all information from publication pending a thorough investigation of the instrumentation by some agency satisfactory to all parties for the purpose of developing complete knowledge concerning the accuracy of the instruments.

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[1] This report is a summary of a detailed report on a cooperative investigation by the U. S. Bureau of Standards and the U. S. Bureau of Public Roads. The purpose of this investigation was to determine the accuracy and other characteristics of such instruments as had been used, or were proposed for use, in the cooperative motor truck impact investigation conducted by the Bureau of Public Roads, the Rubber Association of America, and the Society of Automotive Engineers.

The work was actively guided by a committee composed of members of the technical staff of each bureau as follows.

U. S. Bureau of Standards: Dr. H. C. Dickinson, Dr. L. B. Tuckerman, Mr. E. F. Mueller, and Mr. W. A. Jacobs.

U. S. Bureau of Public Roads: Messrs. E. F. Kelley, L. W. Teller, J. A. Buchanan, and G. P. St. Clair.

Dr. B. Liebowitz was retained as research consultant throughout the investigation.

The actual conduct of the tests was largely the work of Mr. Jacobs and Mr. St. Clair, while the analysis of the data and the preparation of the detailed and summary reports were carried out by Mr. Buchanan and Mr. St. Clair. The mathematical work involved in the investigation, relative to the coil spring accelerometer, the displacement time recorder, and the phenomena of motor truck impact, was prepared by Mr. St. Clair.

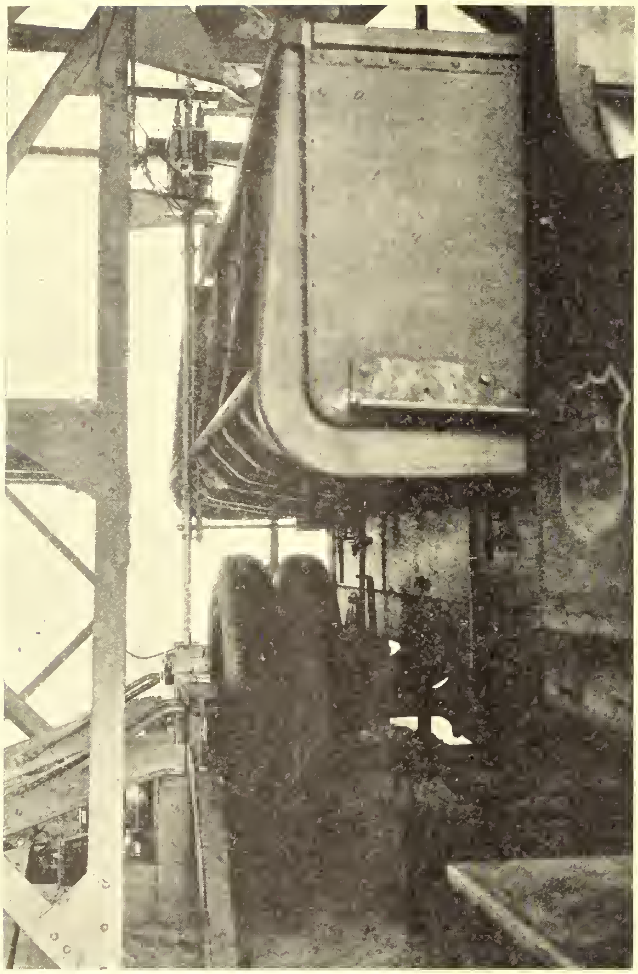

Figure 1.—General View of Truck and Instrument Arrangement for Reaction Wheel Tests

An agreement to conduct a cooperative accelerometer investigation was made between the United States Bureau of Standards and the United States Bureau of Public Roads in April, 1928. This agreement provided for carrying on a program of calibration of accelerometers which had been used or proposed for use in the motor-truck impact research. This investigation has been completed and a detailed report written. The calibration data developed by the Bureau of Public Roads have been incorporated in the report and constitute the section dealing with the reaction wheel tests. Space is not available for the publication of the entire detailed report. The following summary report has been prepared to acquaint the reader with the general nature of tests and analyses made, which form the basis for the conclusions reached.

THE MACHINE FOR PRODUCING SIMPLE HARMONIC MOTION DESCRIBED

It was decided by the committee that the instruments to be investigated should be calibrated on a device which could be made to produce accelerations of known magnitude. For this purpose the Bureau of Standards obtained the loan of a machine for reproducing simple harmonic motion which had been designed and built by the Firestone Tire & Rubber Co. The rotating parts of this machine consist of two flywheels on a 3-inch shaft supported by large bearings in a cast-iron frame. The combined weight of flywheels and shaft is about 1,000 pounds. A driving pulley is attached to one wheel and an adjustable mounting for an eccentric pin is provided on the other wheel. The eccentric pin floats in the yoke of a cross-head frame in such a manner that it is capable of transmitting only the vertical component of its motion. Practically all of the reciprocating parts are of aluminum alloy, their total weight, exclusive of instruments, being about 30 pounds. The machine is belt driven by an electric motor and an independent high-pressure oil system is provided to lubricate all bearings. All bearings are adjustable, so that minimum clearances may be maintained. The rated capacity of the machine is 1,800 revolutions per minute and 4-inch stroke. Tests were run at speeds ranging from 300 to 2,100 revolutions per minute and with strokes ranging from 0.1 to 3 inches.

The speed of the machine was measured by means of a Veeder liquid tachometer, connected by a chain and sprokets directly to the main shaft between the two flywheels. It was calibrated by the Bureau of Standards prior to these tests, and the speed could be determined to the nearest revolution per minute without difficulty. For convenience in making minor changes in speed the machine was equipped with a small hand-brake bearing on the periphery of one of the flywheels.

The reciprocating yoke is guided in a housing which covers the flywheel and cross-head assembly, An aluminum platen about 10 inches square is screwed on the upper end of a tubular extension of the yoke protruding above the housing. Instruments to be tested were mounted on this platen. The stroke of the machine was measured by means of a micrometer depth gage. Figures 4, 8, and 17 show the various instruments mounted for test.

REACTION WHEEL USED TO PRODUCE IMPACT

In the summer of 1926 the Bureau of Public Roads purchased a steam engine flywheel having a diameter of 6 feet 6½ inches and a 19-inch face. This wheel was mounted on a 6-inch shaft carried by a massive reinforced concrete emplacement and furnished an excellent means for conducting motor-truck impact tests under well-controlled laboratory conditions. The pit in which the reaction wheel is situated is covered with a heavy timber floor so that a test truck can be driven into position with one rear wheel resting on the surface of the reaction wheel directly above the center of its shaft. The front axle of the truck is restrained from any motion whatever and the left rear wheel is prevented from moving in the fore-and-aft or lateral directions by means of chocks. The truck wheel is driven in the ordinary manner with a resulting rotation of the reaction wheel. Various obstructions were bolted to the reaction wheel to create impact conditions. To avoid interference between successive impacts, only one obstruction was in use during any given test. Figure 1 shows a test truck mounted on the reaction wheel and Figure 2 shows a closer view of the instrument set-up.

The vertical motion of the truck wheel is transferred to the instrument mounting in the following manner. A hub extension is securely bolted to the wheel and revolves in a self-aligning bearing which is encased in a rectangular bronze block. This block moves with practically no clearance within a steel yoke, which in turn moves freely in vertical guides. The bronze block

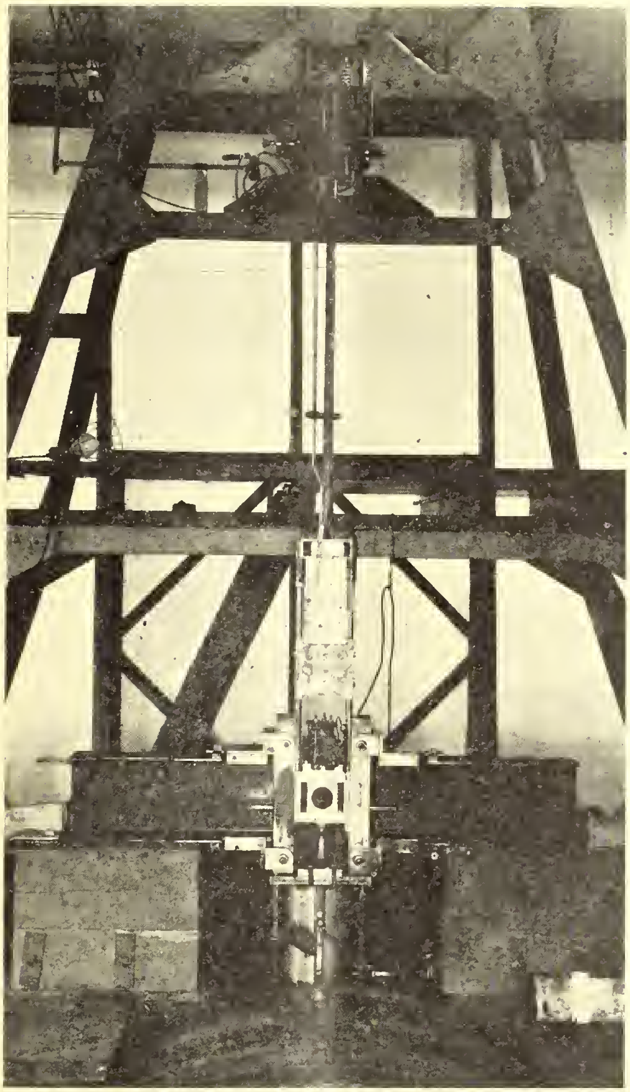

Figure 2.—Installation of Instruments for Reaction Wheel Tests

may move in and out and forward and back, transmitting only the vertical component of the motion of the wheel to the yoke. As may be seen on the cover page, the yoke is a part of the rigid mounting which carries the three instruments used in the tests—the displacement-time recorder, the contact accelerometer, and the coil spring accelerometer. The displacement-time instrument is connected to the base of the yoke, while the contact instrument is mounted directly above it. A rod extending above the truck body carries the coil spring accelerometer, which is restrained from lateral motion by vertical guides. Various devices which are not described here were installed to obtain the semiautomatic control of the instruments which was necessitated by the rapidly repeated impacts.

The two trucks used in tests on this reaction wheel were those which had been used in conducting the major portion of the motor-truck impact tests and the method of loading was the same as in the road tests. The tire equipments used on the trucks were selected from and were representative of those used in the earlier tests. Static load-deflection curves for these tires are given in Figure 3, and Table 1 contains data concerning principal dimensions of tires and weights.

Table 1.—*Tire and weight data*

No.	Type	Size	Over-all section height	Weight (one tire)	Capacity (one tire)	Wheel weight (dual tires)	Unsprung rear wheel weight
		Inches	*Inches*	*Pounds*	*Pounds*	*Pounds*	*Pounds*
47a	Pneumatic (underinflated)	36 by 6	[1] 7⅛	[1] 118	2, 200	435	740
47	Pneumatic (standard inflation)	36 by 6	[1] 7⅛	[1] 118	2, 200	435	740
7	New cushion	36 by 7	4⅛	200	3, 500	792	1, 380
37	New solid	36 by 6	3⅛	168	4, 200	636	1, 224
41a	Worn solid (1 inch visible rubber)	36 by 4	1¾	77	2, 000	461	766
39f	Worn solid (0.6 inch visible rubber)	36 by 4	1⅜	72	2, 000	446	751

[1] Includes demountable rim.

Table 2.—*Schedule of reaction wheel tests completed*

Tire	Truck	Wheel load	Shape and size of obstruction in inches	Speed range, miles per hour
		Pounds		
Dual pneumatic (underinflated).	B	3, 500	1 by 3, rectangular	8 to 20
Dual pneumatic (standard inflation).	B	3, 500	½ by 3, rectangular	8 to 31
			¾ by 3, rectangular	8 to 25
			1 by 3, rectangular	8 to 25
			1½ by 3, rectangular	8 to 20
			1½ by 3, rounded	8 to 19
		5, 000	½ by 3, rectangular	8 to 25
			1 by 3, rectangular	8 to 20
			1½ by 3, rounded	10 to 16
Dual cushion (new)	A	[1] 3, 300	¾ by 3, rectangular	8 to 28
		7, 500	½ by 3, rectangular	7 to 25
			¾ by 3, rectangular	6 to 25
			1 by 3, rectangular	7 to 27
			1½ by 3, rectangular	7 to 15
			1½ by 3, rounded	8 to 22
		10, 000	½ by 3, rectangular	7 to 25
			¾ by 3, rectangular	4 to 25
			1 by 3, rectangular	5 to 25
			1½ by 3, rectangular	6 to 15
			1½ by 3, rounded	5 to 15
Dual solid (new)	A	7, 500	½ by 3, rectangular	8 to 20
			¾ by 3, rectangular	8 to 20
			1 by 3, rectangular	7 to 25
			1½ by 3, rounded	8 to 14
		10, 000	½ by 3, rectangular	8 to 20
			¾ by 3, rectangular	8 to 20
			1 by 3, rectangular	8 to 16
			1½ by 3, rounded	8 to 12
Dual worn solid (1 inch visible rubber).	B	3, 500	½ by 3, rectangular	9 to 17
			¾ by 3, rectangular	9 to 14
Dual worn solid (0.6 inch visible rubber).	B	3, 500	½ by 3, rectangular	8 to 14

[1] Empty truck.

TEST PROGRAM AND SCHEDULE OF COMPUTATIONS DISCUSSED

Table 2 is a schedule of the reaction wheel test program as completed. Tests were made with each of the six tire types, ranging from underinflated pneumatics to badly worn solids. The heavy duty truck, A, was used for the tests with cushion and new solid tires; the lighter truck, B, was used for the pneumatic and worn solid tire tests. Because of the enormous amount of computation involved, it was necessary to select representative tests for analysis, so as to cover as wide a range of conditions as possible. Table 3 is a schedule of tests for which the data were analyzed in full. This analysis included computations of acceleration from the records of the contact accelerometer and of the displacement-time instrument, computation of impact period from the displacement-time record,

Table 3.—*Schedule of reaction wheel tests analyzed in full*

Tire	Truck	Wheel load	Shape and size of obstruction in inches	Type of impact	Number of tests	Speed range, miles per hour
		Pounds				
Dual pneumatic (underinflated).	B	3,500	1 by 3, rectangular	Shock	6	8 to 19
				Drop	6	8 to 20
Dual pneumatic (standard inflation).	B	3,500	½ by 3, rectangular	Shock	1	20
				Drop	1	19
			1 by 3, rectangular	Shock	7	8 to 25
				Drop	7	8 to 25
		5,000	1 by 3, rectangular	Shock	6	8 to 21
				Drop	6	8 to 20
Dual cushion (new).	A	7,500	1 by 3, rectangular	Shock	5	10 to 27
				Drop	6	9 to 26
		10,000	½ by 3, rectangular	Shock	5	12 to 27
				Drop	3	7 to 20
			¾ by 3, rectangular	Shock	9	7 to 24
				Drop	5	11 to 26
			1 by 3, rectangular	Shock	8	5 to 24
				Drop	7	4 to 25
			1½ by 3, rectangular	Shock	3	4 to 14
				Drop	4	8 to 14
			1½ by 3, rounded	Drop	3	4 to 14
Dual solid (new)	A	7,500	½ by 3, rectangular	Shock	1	20
				Drop	1	19
			1 by 3, rectangular	Shock	13	8 to 23
				Drop	10	7 to 26
		10,000	1 by 3, rectangular	Shock	4	7 to 14
				Drop	4	8 to 13
Dual worn solid (1.0 inch visible rubber).	B	3,500	½ by 3, rectangular	Shock	4	10 to 18
				Drop	5	10 to 18
			¾ by 3, rectangular	Shock	4	9 to 15
				Drop	4	10 to 14
Dual worn solid (0.6 inch visible rubber).	B	3,500	½ by 3, rectangular	Shock	3	8 to 12
				Drop	3	9 to 14

Table 4.—*Schedule of reaction wheel tests analyzed for period only*

Tire	Truck	Wheel load	Shape and size of obstruction in inches	Type of impact	Number of tests	Speed range, miles per hour
		Pounds				
Dual pneumatic (standard inflation).	B	3,500	½ by 3, rectangular	Shock	3	8 to 22.
				Drop	2	10 and 31.
			¾ by 3, rectangular	Shock	3	8 to 26.
				Drop	3	8 to 26.
			1½ by 3, rectangular	Shock	3	8 to 21.
				Drop	3	8 to 20.
			1½ by 3, rounded	Shock	3	8 to 18.
				Drop	3	8 to 19.
		5,000	½ by 3, rectangular	Shock	3	8 to 20.
				Drop	3	8 to 24.
			1½ by 3, rounded	Shock	3	10 to 16.
				Drop	3	10 to 15.
Dual cushion (new).	A	[1] 3,330	¾ by 3, rectangular	Shock	11	8 to 27.
				Drop	7	8 to 27.
		7,500	½ by 3, rectangular	Shock	6	8 to 18.
				Drop	5	7 to 25.
			¾ by 3, rectangular	Shock	7	8 to 23.
				Drop	11	8 to 22.
			1 by 3, rectangular	Shock	1	9.
			1½ by 3, rectangular	Shock	4	7 to 14.
				Drop	4	8 to 15.
			1½ by 3, rounded	Shock	5	8 to 21.
				Drop	4	8 to 20.
		10,000	½ by 3, rectangular	Shock	4	7 to 20.
				Drop	6	7 to 27.
			¾ by 3, rectangular	Shock	6	5 to 26.
				Drop	10	5 to 24.
			1 by 3, rectangular	Shock	2	15 and 20.
				Drop	6	5 to 24.
			1½ by 3, rectangular	Shock	4	8 to 14.
				Drop	4	4 to 14.
			1½ by 3, rounded	Shock	5	4 to 15.
				Drop	2	11 and 15.
Dual solid (new)	A	7,500	½ by 3, rectangular	Shock	2	8 and 14.
				Drop	2	8 and 14.
			¾ by 3, rectangular	Shock	3	8 to 21.
				Drop	3	8 to 21.
			1½ by 3, rounded	Shock	3	8 to 14.
				Drop	3	8 to 14.
		10,000	½ by 3, rectangular	Shock	3	8 to 19.
				Drop	3	8 to 20.
			¾ by 3, rectangular	Shock	3	8 to 19.
				Drop	3	8 to 20.
			1½ by 3, rounded	Shock	3	8 to 12.
				Drop	3	8 to 11.

[1] Empty truck.

measurement of records from the coil spring accelerometer, and application of a method of correlation between the coil spring accelerometer and the other two instruments. Table 4 is a schedule of tests for which computations of impact periods were made by analysis of displacement-time records. The technique of the analyses and the data obtained will be described in subsequent paragraphs.

METHOD OF ANALYSIS DESCRIBED

The reaction wheel tests were conducted primarily to calibrate the coil spring accelerometer under the conditions of motor-truck impact. The displacement-time recorder and the contact accelerometer were used as control instruments, and the analysis of the data obtained from the coil spring accelerometer is based upon the data obtained from the other two instruments.

This report is concerned chiefly with the question of the accuracy of the three instruments used for measuring accelerations. In considering the error and dispersion of a given instrument or the agreement between two instruments, it has been found necessary to employ certain terms and methods analogous to those used in statistical work without adopting the rigorous procedure of that science. An error or a deviation from a mean value is generally expressed as a percentage of the correct or mean value. Thus if A_M is a mean or assumed correct value of acceleration and A_e is a corresponding value deviating from it, the percentage of error is defined by the equation $e = 100\left(\frac{A_e}{A_M} - 1\right)$. The value of e is generally given absolute without regard to sign.

When data showing the relation between two variables exhibit considerable dispersion, a frequency diagram is substituted for the customary plot of coordinates. The frequency diagram is divided into squares, the length of the sides of each square representing convenient intervals in the two variables involved. Within each square is written a number equal to the number of points which would appear within that square if the coordinates were plotted. In this manner the distribution of the points about a mean position is brought out clearly. Figures 12, 18, 20, 22, and 25 are examples of such frequency diagrams.

In a number of cases it was desirable to develop functions expressing the probable error of the data. This was done by drawing curves in such a manner that half the data shown in the given frequency diagram lies below and half above the curve. A similar curve is drawn bounding a zone which includes 90 per cent of the data. These curves are denoted by the symbols e_{50} and e_{90} as in Figures 13, 21, and 26.

THE DISPLACEMENT-TIME RECORDER DESCRIBED

It is possible, with the reaction wheel set-up, to obtain an autographic record of the vertical motion of a truck wheel under impact. If such a record is taken on a drum rotating at a known constant speed, a curve is obtained from which the acceleration-time

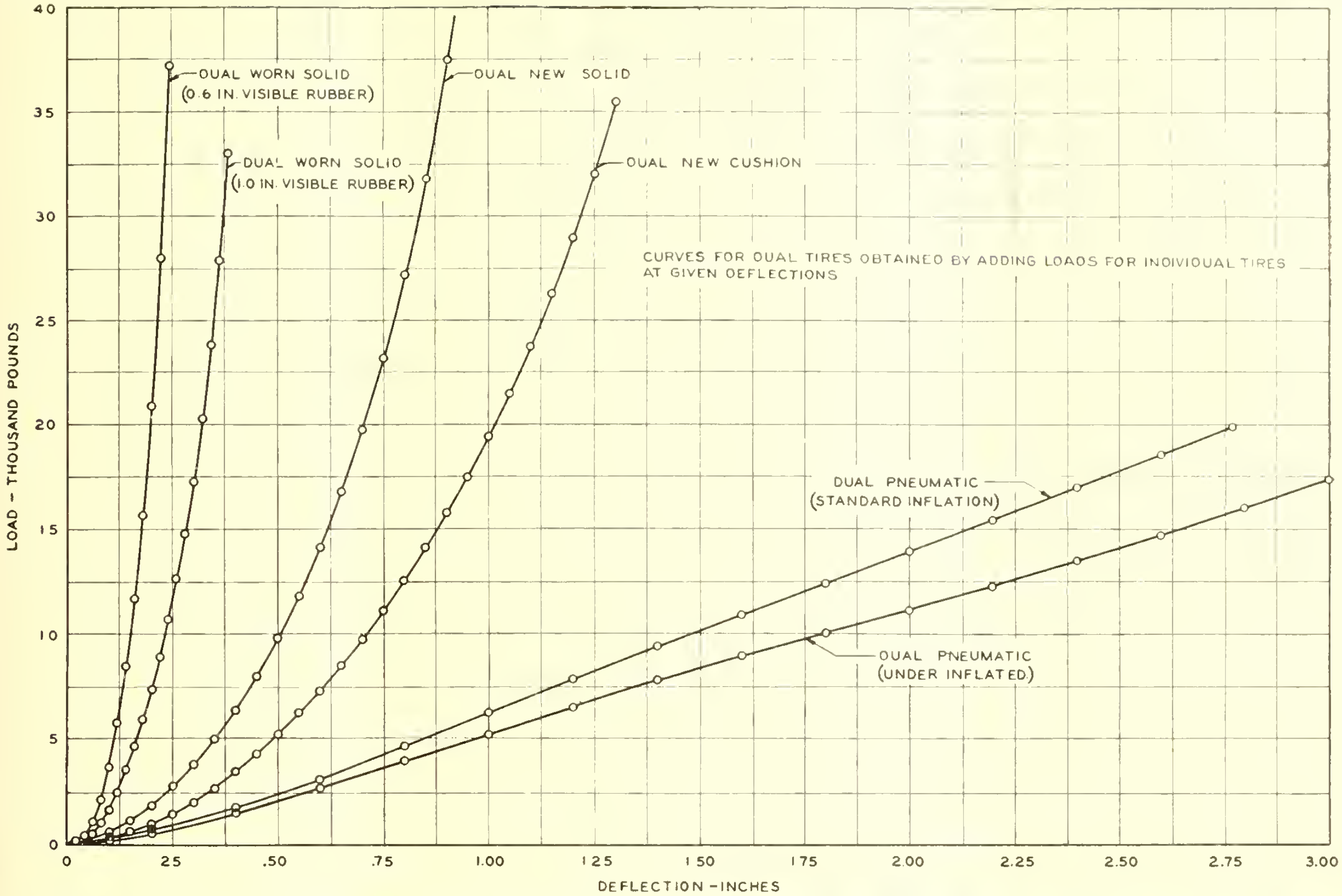

Figure 3.—Static Load-Deflection Curves for Tires used in Reaction Wheel Tests. Curves for Dual Tires Obtained by Adding Loads for Individual Tires at Given Deflections

relation may be derived by differentiation. The Bureau of Public Roads has used this method of measuring acceleration for a number of years. The early calibration of the coil spring accelerometer was based on displacement-time tests conducted with a stationary drop-impact machine. A new instrument was constructed for use in the reaction-wheel tests. Figure 4 shows this instrument mounted to record the vertical movement of the yoke on the simple harmonic motion machine at the Bureau of Standards. The cover page shows its installation for the reaction-wheel tests.

The essential elements of the displacement-time recorder are, (1) a vertically moving rod which carries a recording stylus, (2) a drum which revolves at a constant speed about a vertical axis, and (3) a direct-recording tuning fork. The rod moves in specially constructed adjustable bronze bearings which are lapped in so as to allow free movement with minimum clearance. The rod is connected to the body whose motion it is desired to record by a short stud having a necked section one-eighth inch in diameter and one-half inch in length. This flexible link was designed to eliminate binding of the rod in its bearings due to slight lateral displacements of the moving body, and, in case of accident, to serve as a weak link protecting the instrument from violent distortion. The recording stylus is mounted within a solenoid and is moved into contact with the paper by the operation of a key switch.

The drum is of aluminum and is approximately 7¼ inches in diameter and 9½ inches in height. It turns in conical bearings, and is driven by a small motor through a 100 to 1 worm-gear reduction. The motor speed is about 10,000 revolutions per minute, producing a tangential velocity of approximately 38 inches per second at the surface of the drum. The record paper is attached to the face of the drum with rubber cement and is coated with a light film of smoke. Upon removal from the drum the record is fixed by being dipped in a thin solution of pure white shellac in grain alcohol.

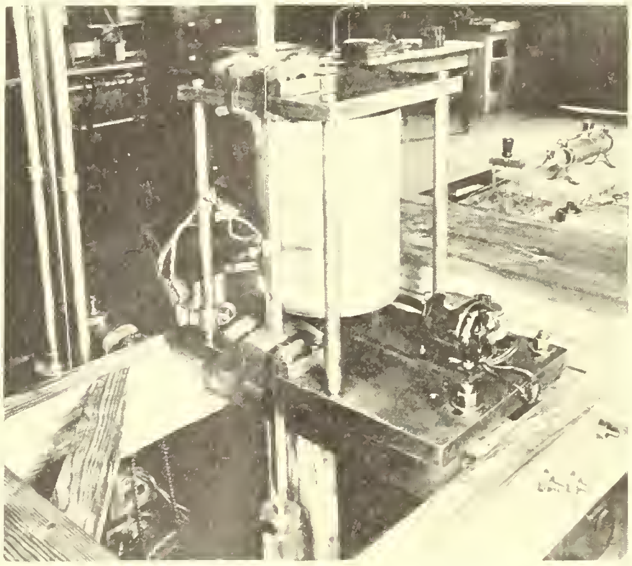

Figure 4.—Displacement-Time Apparatus on the Simple Harmonic Motion Machine

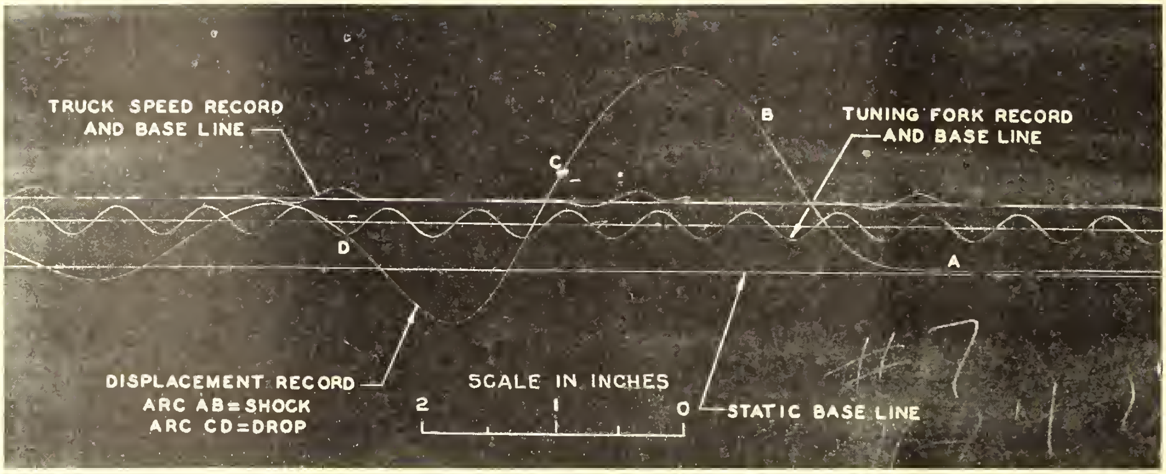

Figure 5.—Photograph of Displacement-Time Record

The tuning fork is driven by an electromagnet operating on a make-and-break circuit. The record is made by a small stylus of very thin brass projecting from the end of one prong and resting very lightly on the record paper. The tuning fork has a frequency of approximately 58 cycles per second.

The accuracy with which accelerations can be determined depends largely upon the precision with which the coordinates of displacement and of time can be measured. It is therefore necessary that the amplitude and period of the motion to be analyzed be large with respect to the errors in the measurement of these quantities. The accuracy of the determination depends further upon the nature of the motion involved and it may be stated as a general rule that the curve to be analyzed should be the record of a fundamental motion of great regularity or "smoothness," disturbed only slightly if at all by parasitic vibrations or irregularities.

Tire deformations under impact are generally of considerable amplitude, varying from a few tenths of an inch in the case of worn solid tires to an inch or more in the case of pneumatic tires. The periods of impact reaction vary from less than 0.01 second for badly worn solids to as much as 0.10 second for pneumatics. In almost all cases the displacement-time curves are of a regular character, exhibiting a gradual change of curvature with the passage of time. Small parasitic oscillations, however, are always present. These oscillations may be due to engine vibrations, to small local structural deformations, or to disturbances within the recording mechanism itself. They cause minute irregularities in the curve, and are eliminated in the analysis by a process of smoothing. It may be stated that, in general, displacement-time records of impact reactions may be accurately analyzed.

However, because of the wide range of amplitudes and periods and the varying effect of parasitic irregularities, the accuracy is necessarily variable. The most uniform results were obtained from tests with cushion and new solid tires. Tests with worn solid tires were rather unsatisfactory because of the extremely violent reactions occurring with such tires.

SOURCES OF ERROR IN RECORDING ANALYZED

In the complete report of these investigations there is given a detailed discussion of the errors to which the displacement-time recording mechanism is subject. The following sources of error are considered, and estimates are made of their probable effect, based on such data as are available:

(1) Sources of error in displacement only.
  a. Elastic deformations of the vertically moving rod and its flexible connection to the body whose motion is being measured.
  b. Movement of the supports on which the instrument is mounted.
(2) Sources of error in time only.
  a. Error in calibration of the tuning fork.
  b. Irregularities in the functioning of the tuning fork.
  c. Fluctuation in speed of the drum.
(3) Sources of error in both displacement and time.
  a. Errors in alignment and verticality of the rod and the drum.
  b. Eccentricity of the drum.
  c. End play in the drum.
  d. Irregularities in the record paper and in the manner of its mounting on the drum.
  e. Malfunctioning of the recording stylus, due to—
    1. Eccentricity of the point.
    2. Play in the barrel.
    3. Bending of the projecting portion of stylus.
  f. Shrinkage or expansion of the paper after fixing with shellac solution.

It was found that for moderate test conditions, such as those encountered with cushion and solid tires, the probable error in acceleration due to imperfections in the recording device lies between 1 and 2 per cent. For severe conditions, as in the case of tests with worn solid tires, the error is undoubtedly much greater. For highly favorable conditions it may be much less.

METHOD OF DIFFERENCES USED IN ANALYZING RECORDS

For use in analyzing displacement-time curves a special comparator was built to order. By means of this comparator one may read rectangular coordinates, over a 4-inch range for each coordinate axis, directly in ten-thousandths of an inch. It was calibrated by the Bureau of Standards and found to be highly accurate. Errors inherent in the comparator are entirely negligible in comparison with the errors of the recording device, discussed in the preceding paragraph.

Figure 5 is a photograph of a displacement-time curve obtained in the reaction-wheel tests. The significant portions, or arcs, of the curve are marked "shock" and "drop." The former occurs as the tire encounters and mounts an obstruction, the latter as it again reacts upon the surface of the reaction wheel. The object of the analysis is to obtain for either or both of these arcs a curve or tabulation of values expressing the variation of acceleration corresponding to the displacement-time variation given by the arc in question.

For the desired portion of the curve comparator readings of displacement are made at regular intervals of the time coordinate and tabulated. The method used in analysis is numerical rather than graphical and is based on the calculus of differences.[2] This method is described in full with examples in the detailed report. The first step in the analysis is the elimination, by the process known as "smoothing," of irregularities caused by parasitic oscillations of high frequency or by irregularities in the width of the record line. This is accomplished by replacing the series of displacement coordinates given by the comparator readings with a smooth series differing from it only by very small amounts. It has been found in practice that in almost all cases the smoothing can be restricted to changes of less than 0.002 inch in magnitude and that the great majority of changes can be held below 0.001 inch. The fact that the process of smoothing involves changes in the fourth decimal place with occasional changes of 1 or 2 in the third place indicates that the probable errors due to the use of that process are well within the limits of accuracy imposed by the imperfections of the recording apparatus and the varying qualities of the displacement-time curves themselves.

The first differences of a tabulated series are obtained by subtracting each tabulated value from the value next following it. The second differences are obtained by applying the same process to the first differences, and so on. If the series is tabulated for regular intervals of the independent variable or argument (time in this case) values of the derivatives of the tabulated function with respect to the argument may be obtained by the use of formulas involving the tabular differences as defined above. Accelerations were computed by the application of such formulas to the smoothed series of displacement coordinates, tabulated at regular intervals of time.

Given a smooth series, no error is involved in this method of analysis except those "last-place" errors which are characteristic of all computations where values are given in a fixed number of decimal places. These inaccuracies may be eliminated by adding one or more fictitious decimal places and carrying out the process of smoothing until the series is "smooth" in the last added place. Such a procedure does not, of course, affect the true accuracy of the acceleration measurement, which depends on other factors.

DISPLACEMENT-TIME RECORDER TESTED ON SIMPLE HARMONIC MOTION MACHINE

A number of tests were conducted with the instrument mounted as shown in Figure 4 to record the reciprocating movement of the simple harmonic motion machine. Tests were made at the following strokes and speeds:

Stroke, inches (depth gage)	Revolutions per minute (liquid tachometer)
0. 997	299, 599, 899
2. 000	299, 599, 899
2. 998	299, 599, 899

The corresponding periods of cycle, computed by means of the formula, $T_1=\frac{60}{\text{r. p. m.}}$, are 0.2007, 0.1002, and 0.0667 second, respectively.

In each case the downward half-cycle was chosen for analysis, because it is analogous to the "drop" in motor truck impact. Values of acceleration were obtained by three different methods, and given the symbols A_1, A_2, and A_3. They are defined as follows:

$$A_1=\frac{4\pi^2}{T_1^{\,2}}a,$$

where a is one-half the stroke as measured by the depth gage and $T_1=\frac{60}{\text{r. p. m.}}$ as measured by the liquid tachometer.

$$A_2=\frac{4\pi^2}{T_1^{\,2}}a,$$

where a is one-half the stroke and T_1 is twice the period of the lower half-cycle, both quantities being obtained by lineal measurements on the displacement-time record.

A_3 = acceleration obtained from analysis of the displacement-time record by the method of differences.

The value of A_1 is independent of the displacement-time record and may be regarded as the basic value with which A_2 and A_3 are to be compared. The accuracy of A_2 depends upon the precision with which displacement and time are recorded and measured. In the case of A_3 the method of analysis, including the process of smoothing, is involved.

Table 5 gives the values of A_1, A_2, and A_3 for the 16 curves which were analyzed. In Figure 6 the following combinations are plotted: A_2 against A_1, A_3 against A_2, and A_3 against A_1. Lines are drawn having a 1 to 1 slope to denote complete agreement.

In order to indicate the extent to which the various values of acceleration deviate from complete agreement, values of the following quantities are given in Table 6:

$$A_2-A_1,$$

$$e_2=100\left(\frac{A_2}{A_1}-1\right),\ \text{(absolute value)},$$

$$A_3-A_1,$$

$$e_3=100\left(\frac{A_3}{A_1}-1\right),\ \text{(absolute value)}.$$

It is possible, without resort to statistical analysis, to obtain a fair idea of the probable errors of A_2 and A_3 on the assumption that A_1 is correct. By inspection of Table 6 we find that the median values of e_2 and e_3 are as follows:

Median $e_2=0.98$ per cent.

Median $e_3=1.07$ per cent.

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[2] A discussion of this subject is given in the Encyclopedia Britanica, eleventh edition, under the heading "Differences, Calculus of." A more complete treatment is contained in "The Theory and Practice of Interpolation," by H. L. Rice, the Nichols Press, 1899.

TABLE 5.—*Values of A_1, A_2, and A_3 for the 16 curves analyzed*

Test No.	Stroke	Speed	A_1	A_2	A_3
	Inches	*Revolutions per minute*	*Feet per second per second*	*Feet per second per second*	*Feet per second per second*
1	2.000	299	81.7	82.5	83.0
3	2.000	599	327.9	329.3	327.7
4	2.000	599	327.9	329.5	330.3
5	2.000	899	738.5	735.4	770.6
6	2.000	899	738.5	750.9	756.6
7	2.998	299	122.4	124.9	125.4
10	2.998	599	491.5	491.6	495.4
11	2.998	599	491.5	496.8	491.1
15	2.998	899	1,107.1	1,111.1	1,118.9
16	.997	299	40.7	41.1	40.9
17	.997	299	40.7	41.7	42.4
18	.997	599	163.4	164.9	167.9
19	.997	599	163.4	165.0	163.4
20	.997	899	368.2	371.9	369.2
21	.997	899	368.2	372.4	375.1

TABLE 6.—*Values indicating the extent to which the various values of acceleration deviate from complete agreement*

Test No.	A_1	A_2-A_1	$e_2=100\times\left(\frac{A_2}{A_1}-1\right)$	A_3-A_1	$e_3=100\times\left(\frac{A_3}{A_1}-1\right)$
	Feet per second per second	*Feet per second per second*		*Feet per second per second*	
1	81.7	+0.8	0.98	+1.3	1.59
3	327.9	+1.4	.43	−.2	.06
4	327.9	+1.6	.49	+2.4	.73
5	738.5	−3.1	.42	+32.1	4.35
6	738.5	+12.4	1.68	+18.1	2.45
7	122.4	+2.5	2.04	+3.0	2.45
10	491.5	+.1	.02	+3.9	.79
11	491.5	+5.3	1.08	−.4	.08
15	1,107.1	+4.0	.36	+11.8	1.07
16	40.7	+.4	.98	+.2	.49
17	40.7	+1.0	2.46	+1.7	4.18
18	163.4	+1.5	.92	+4.5	2.75
19	163.4	+1.6	.98	0	0
20	368.2	+3.7	1.00	+1.0	.27
21	368.2	+4.2	1.14	+6.9	1.87

The median is, by definition, a value such that the number of values exceeding it is equal to the number of values which it exceeds. The median values of e_2 and e_3 may be taken therefore as approximate probable errors of A_2 and A_3, respectively.

It may be observed that these values are of the same order of magnitude as that of the probable error obtained from an analysis of the sources of error inherent in the recording mechanism.

DISPLACEMENT-TIME RECORD TAKEN ON ALL REACTION WHEEL TESTS

A displacement-time record was taken in each test conducted on the reaction wheel. As is shown in Figure 5, each curve so obtained contains both shock and drop segments. However, in any given test, the contact instrument was set to record either shock or drop as desired. In consequence, the analysis of the displacement-time record was confined to the segment corresponding to that for which the setting of the contact instrument was made. Both the acceleration A_{DT}, and the period of impact T_1 were obtained in the analysis. The term T_1, as used in motor-truck impact tests, is defined as twice the duration of the upward acceleration, in order to obtain a value analogous to the period of a complete cycle in simple harmonic motion. In addition, the amplitude of the impact displacement was measured for use in computing the acceleration A_C as determined by the contact accelerometer.

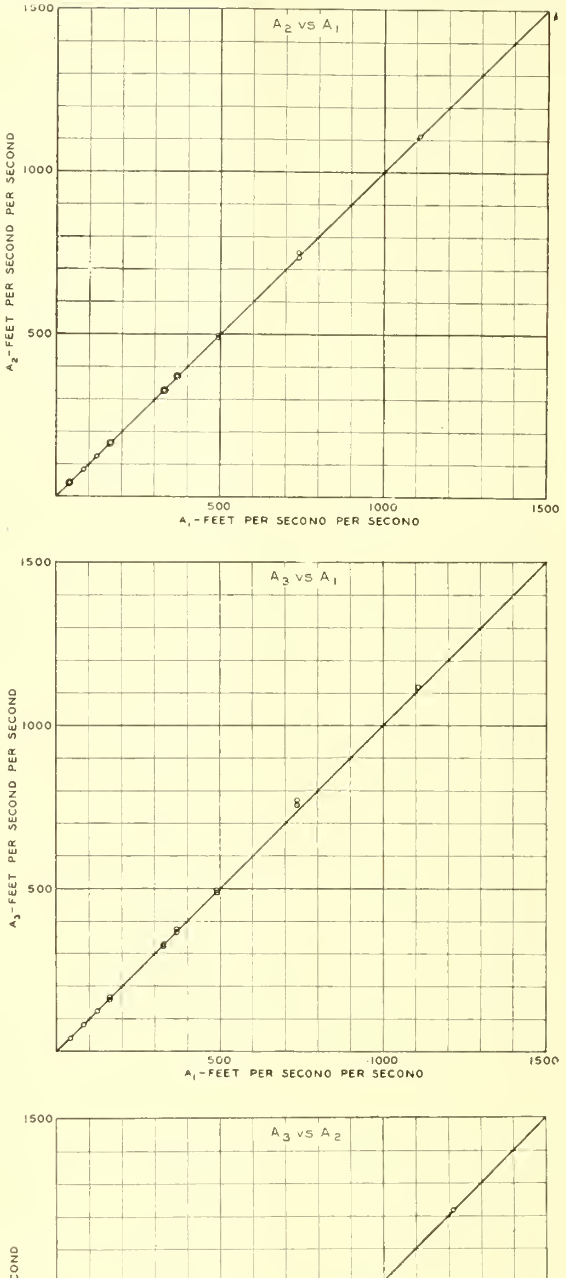

FIGURE 6.—TESTS OF DISPLACEMENT-TIME RECORDER ON SIMPLE HARMONIC MOTION MACHINE. COMPARISON OF ACCELERATIONS COMPUTED BY THREE METHODS: A_1: STROKE MEASURED BY MEANS OF DEPTH GAGE, PERIOD BY MEANS OF LIQUID TACHOMETER; A_2: STROKE AND PERIOD OBTAINED FROM MEASUREMENTS ON DISPLACEMENT-TIME RECORD; A_3: ACCELERATION OBTAINED BY METHOD OF DIFFERENCES

THE CONTACT ACCELEROMETER DESCRIBED

The contact accelerometer[3] is represented diagrammatically in Figure 7. The sensitive element is pivoted at O, to the left of which extends a flat cantilever spring S of uniform section and to the right of which is an inertia element W. The spring is deflected and the deflection is measured by means of a micrometer M. The motion of the weight W is limited by the opposed contact screws BC and MC. A suitable rigid frame carries the barrel of the micrometer M, the pivot O, and the contact screws BC and MC in fixed relation to one another.

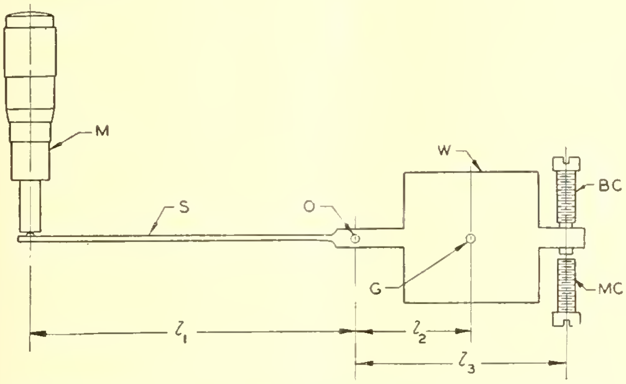

Figure 7.—Diagram of Micrometer-Type Contact Accelerometer

When the spring S is deflected downward by the micrometer M, the weight W is pressed upward against the contact screw BC with a force determined by the load-deflection rate of the spring and the deflection imposed upon it. Now, if the weight be subjected to an acceleration acting downward with respect to it (or if the frame be subjected to an acceleration acting upward with respect to the weight), the weight will move from contact screw BC and bear against contact screw MC when the magnitude of the acceleration acting upon its mass creates a force sufficient to overcome the resistance of the deflected spring. By varying the spring deflection the critical acceleration at which this takes place is varied, and the spring deflection at the micrometer is a measure of such acceleration. Whether or not these contacts are broken or made is determined electrically by means of a telephone headset.

When the circuit is arranged to detect "break" at the upper contact screw BC, the instrument is so sensitive as to be practically useless in measuring accelerations due to motor-truck impact because of the high-frequency, low-amplitude vibrations which are nearly always parasitic to such impact reactions. However, when conditions are such that the break circuit can be used the micrometer deflection beyond the break zero setting can be used without resort to correction factors to determine the critical acceleration. With the break circuit no knowledge is required concerning the amplitude or period of the impressed acceleration, nor is the gap between the break and make contacts of importance.

When parasitic vibrations are present, particularly when such vibrations are unimportant with respect to the basic accelerations to be measured, their effect may be eliminated by arranging the telephone circuit to detect contact at the lower or "make" contact screw MC, using a suitable "gap" clearance (as defined below). The sensitivity of the instrument, however, decreases with the magnitude of the gap opening (defined as the movement of the weight between the make and break contacts). It has been found that a gap of 0.002 inch at the center of gravity of the weight G is ordinarily sufficient to avoid interference by such parasitic vibrations. Since this movement corresponds to nearly 0.006 inch movement at the micrometer to cause the weight to move between the upper and lower contact screws, the uncorrected use of the "make" zero setting of the micrometer could, at times, result in noticeable error in the magnitude of the indicated critical acceleration. This error depends upon the relation between the gap clearance and the amplitude of the impressed acceleration. A theoretical discussion of this effect may be found in the November, 1925 issue of the Journal of the Society of Automotive Engineers (pp. 433–435). Empirical correction factors determined under conditions of simple harmonic motion are given in a subsequent paragraph.

The instrument is calibrated statically by determining the load-deflection rate s of the spring and determining the mass effective as at the center of gravity. It is convenient to express the load-deflection rate in pounds (applied at the center of gravity of the weight) per inch of deflection (measured at and by the micrometer). This is done by suspending known weights at the point G and measuring the micrometer intervals required to restore contact BC or to break contact MC, as the case may be. The mass m of the weight and the position of the center of gravity are obtained by computation. The calibration rate C in feet per second per second per inch may be determined from the equation,

$$C=\frac{s}{m}$$

DYNAMIC CALIBRATION TESTS MADE ON SIMPLE HARMONIC MOTION MACHINE

Dynamic calibration tests were made on the machine for producing simple harmonic motion which had been installed at the Bureau of Standards. The ranges in stroke, speed, and acceleration of the machine corresponded to the amplitude, period, and acceleration of motor truck impact encountered through a range in tire equipment from pneumatic to partly worn solid tires. Tests were made with the telephone detector of the instrument arranged in both the break (upper contact) and the make (lower contact) circuits. In order to determine the influence of gap clearance when using the make contacts this condition was varied through a fairly wide range.

A general view of the instrument installed on the simple harmonic motion machine is given in Figure 8. With the simple harmonic machine in motion the instrument moved in accordance with the motion of the crosshead and the micrometer could not be read in ordinary light. A neon lamp (shown attached to the extension cord) was arranged to flash at the culmination of each down stroke of the platen, within the shadow of a hood covering the apparatus. (Fig. 8.) The micrometer was illuminated at only one position of its path (stroboscopic effect) and the operator was enabled to read it quite easily at any speed obtained in the tests.

For each test condition, the acceleration was computed from the known values of stroke and speed of the harmonic motion machine. For any given test,

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[3] See Jour. S. A. E., March, 1926, pp. 249–250.

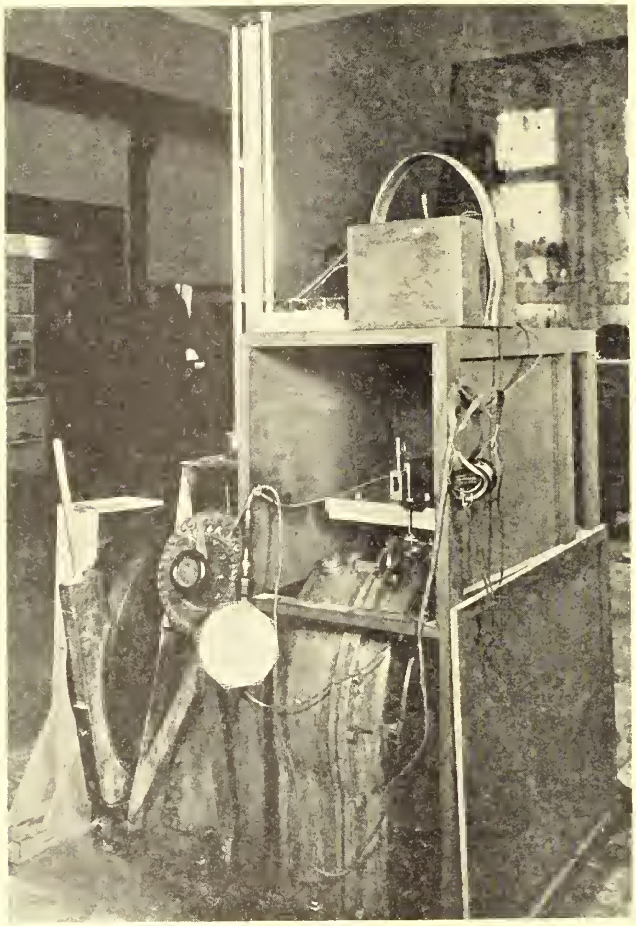


Figure 8.—The Single Element Contact Accelerometer and Neon Lamp Arranged for Tests on the Simple Harmonic Motion Machine

the reading of the micrometer was subtracted from its zero reading to obtain the spring deflection ΔM. The upper or "break" contact can not ordinarily be used in motor-truck impact tests, and it is sufficient to state that in the case of all strokes of from 0.40 to 3.00 inches the points fall very closely about a straight line for each of the two instruments tested. Some apparent discrepancies occurred at strokes of 0.10 and 0.20 inch. The "break" calibration of one of the instruments (No. 1), is given, as an example, in Figure 9, the rate being 5.28 feet per second per second per 0.001 inch.

As stated previously, it has been found that the instrument can be satisfactorily used in motor-truck impact tests when the influence of parasitic vibrations is cut out by a gap of 0.002 inch used in conjunction with the make contact telephone circuit. Calibration tests involving this gap resulted in a series of curves which yield, when the amplitude of a simple harmonic cycle is known, the acceleration corresponding to a given deflection of the accelerometer spring. It is probable but by no means certain that in motor-truck impact the accelerometer is influenced by the amplitude and period in much the same way as it is in simple harmonic motion. On this assumption the curves afford a means of reading acceleration (due to motor-truck impact) directly, if the spring deflection and amplitude of motion are known; and, if the assumption is correct, the accuracy is high (within the limits of the test conditions). The "make" calibration for instrument No. 1, using 0.002-inch gap, is given in Figure 10, which shows curves of constant spring deflection plotted against acceleration and stroke. The circles are points interpolated from work sheets of the test data, the crosses are extrapolated or interpolated points and the dotted lines indicate that the curvature was estimated. It may also be noted that, in general, the "break" calibration rate could also be used under these "make" conditions without introducing serious errors so far as the measurement of acceleration due to motor-truck impact is concerned.

Make calibration tests were also made using gaps of 0.001, 0.004, and 0.007 inch. The curve for the shorter gap adheres somewhat more closely to that for the break calibration, while those for the wider gaps showed greater divergence than the curve for 0.002-inch gap. Since these gaps were not used in subsequent tests by the committee, they have been omitted from this report.

MEASUREMENT OF ACCELERATION BY ANALYSIS OF DISPLACEMENT-TIME CURVES COMPARED WITH MEASUREMENT BY CONTACT ACCELEROMETER

In the reaction wheel tests the accelerations produced were measured by means of both the displacement-time apparatus and the contact accelerometer. In analyzing the results obtained by these two instruments it is not sound to assume that either gives the correct acceleration, and the arithmetical mean between the two values was taken as the most probable acceleration for each test. Some conception of the agreement between the two methods may be had by referring to Figure 11, which contains 154 plotted points.

The difference between the two measurements may be studied by use of a quantity analogous to the percentage of error used in other discussions in this report.

Let A_{DT} = acceleration as given by the displacement-time analysis,

A_C = acceleration as given by the contact accelerometer,

Then $A_M = \dfrac{A_{DT} + A_C}{2}$ = arithmetical mean,

Let $e = 100\left(\dfrac{A_{DT} - A_C}{A_M}\right)$ (absolute value).

The quantity e may be defined as the deviation of one value of acceleration from the other expressed as a percentage of the mean between them. In Figure 12 these data are given in the form of a frequency pattern, the number within each rectangle being the number of points falling between the respective coordinate limits. The size of rectangle varies because of the sparsity of points at high accelerations.

In Figure 13 curves are drawn to represent zones in e, including 50 per cent and 90 per cent of the data. For each column of rectangles in Figure 12 an ordinate was determined such that the frequency of points having a greater value of e is equal to the frequency of points having a less value of e. Similarly, an ordinate was determined such that the ratio of frequencies is nine to one. These two sets of points are plotted in Figure 13, the abscissa of each point being equal to that of the midpoint of the corresponding column of rectangles in Figure 12. The curves e_{50} and e_{90} are drawn through the points so plotted with due regard to the frequencies represented. It will be observed that, except in the case of low accelerations, the e_{50} curve lies in the neighborhood of a value of e equal to 4 per cent, while the e_{90}-curve varies between 8 and 12 per cent.

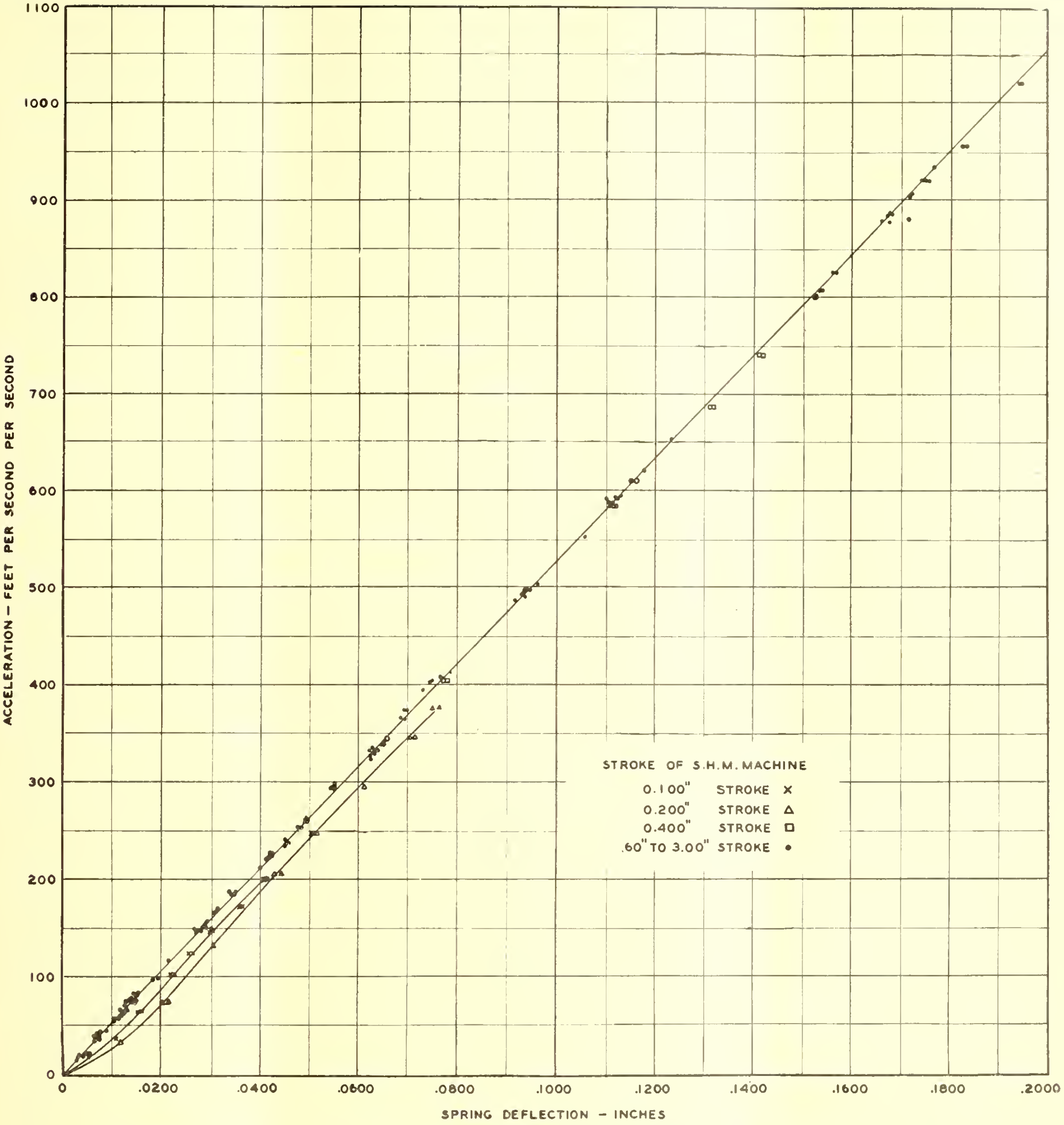


Figure 9.—Calibration of Contact Accelerometer No. 1 Under Break Contact Condition

The e_{50} curve may be regarded as a curve of probable variation between the accelerations given by the two instruments. If e had been computed on the basis of variation from the mean acceleration A_M, the ordinates of the points and the curve determined by them would have been only half as great. If, therefore, it is assumed that A_M is the most probable value of acceleration, it follows that the probable error of either instrument lies between 2 and 3 per cent for a wide range of acceleration values. These facts lead to the conclusion that both the displacement-time recorder and the single element contact accelerometer are satisfactory for the measurement of impact accelerations under laboratory conditions.

THE COIL SPRING ACCELEROMETER DESCRIBED AND THEORY OF OPERATION DISCUSSED

Figure 14 shows a single-element coil spring accelerometer mounted on a test truck for use in road tests. Figure 15 is a diagrammatic representation of such an instrument. The sensitive element consists of a weight mounted on a helical spring. Under the impulse of an upward acceleration the weight moves downward relative to the frame of the instrument, compressing the spring. The relative motion is recorded on sensitized paper which moves horizontally in the frame. In its rest position the spring is under no compression save that due to the weight, and the weight is in contact with a backstop which prevents it from moving

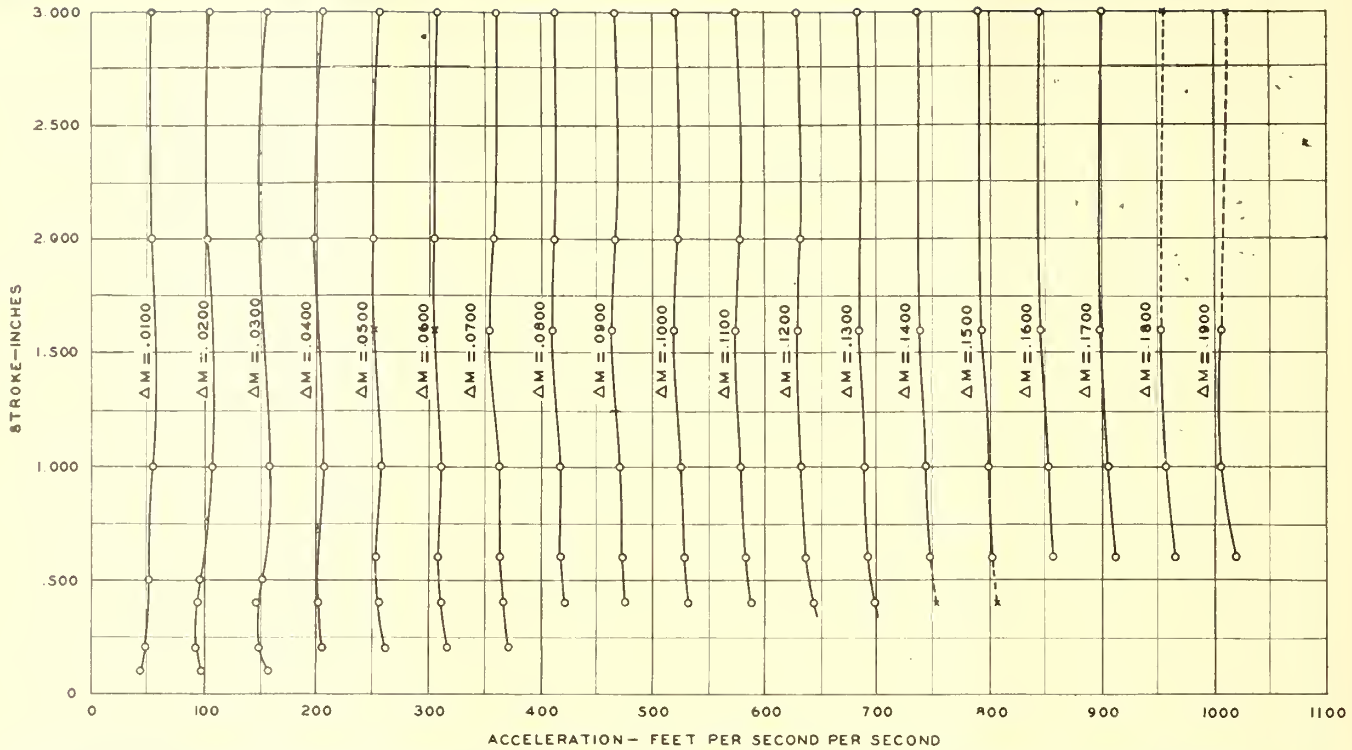


FIGURE 10.—CALIBRATION CURVES FOR CONTACT ACCELEROMETER NO. 1, UNDER CONDITIONS OF MAKE CONTACT AND A CONTACT GAP OF 0.002 INCH AT THE CENTER OF GRAVITY OF THE SENSITIVE ELEMENT. ACCELERATIONS ARE PLOTTED AGAINST STROKE, EACH CURVE REPRESENTING A GIVEN VALUE OF THE SPRING DEFLECTION (ΔM)

upward beyond that position. Because of this backstop the instrument will not respond to a downward acceleration.

The detailed report includes a discussion of the theory underlying the operation of the coil-spring accelerometer. The theory is based on the assumption that the force exerted by the spring is proportional to its deflection. This assumption, which is very nearly correct for springs tested statically, leads to the development of the following approximate formula for the free period of vibration, T_2, of the spring-weight element:

$$T_2 = 2\pi\sqrt{\frac{w}{gs}},$$

where w = the effective weight, taken as the weight resting on the spring plus one-half the weight of the spring,
s = the loading rate of the spring.

The differential equation for the motion of the sensitive element takes the form,

$$\frac{d^2r}{dt^2} = -\frac{d^2y}{dt^2} - \frac{4\pi^2}{T_2^2}r = -\frac{d^2y}{dt^2} - \frac{gs}{w}r \text{ - - - - - - - - } (1)$$

where r = the displacement of the element relative to the frame at time t,
$\frac{d^2y}{dt^2}$ = the acceleration acting on the frame at time t.

In order to obtain a simple expression for the term $\frac{d^2y}{dt^2}$, the assumption was made that the motion of the frame is that of simple harmonic motion and may be expressed by the equation

$$y = a \sin \frac{2\pi}{T_1}t$$

where

y = the displacement of the frame at time t,
a = amplitude of the motion,
T_1 = period of the motion.

On this assumption equation (1) takes the form,

$$\frac{d^2r}{dt^2} = \frac{4\pi^2}{T_1^2}a \sin \frac{2\pi}{T_1}t - \frac{4\pi^2}{T_2^2}r \text{ - - - - - - - - - - - } (2)$$

The initial conditions of the motion are as follows:

$$t=0 \qquad y=0 \qquad r=0$$

$$\frac{dy}{dt} = \frac{2\pi}{T_1}a \qquad \frac{dr}{dt} = 0$$

$$\frac{d^2y}{dt^2} = 0 \qquad \frac{d^2r}{dt^2} = 0$$

The following integral of equation (2) was found to satisfy the conditions governing the operation of the coil spring accelerometer.

$$r = \frac{a}{n^2-1}\left(\sin \frac{2\pi}{T_1}t - \frac{1}{n}\sin \frac{2\pi}{T_2}t\right), \quad \text{Case I - - - - } (3)$$

where

$$n = \frac{T_1}{T_2}.$$

In the special case where $n=1$, i. e. $T_1 = T_2 = T$, the integral takes the form,

$$r = \frac{a}{2}\left(\sin \frac{2\pi}{T}t - \frac{2\pi}{T}t \cos \frac{2\pi}{T}t\right), \quad \text{Case II - - - - } (4)$$

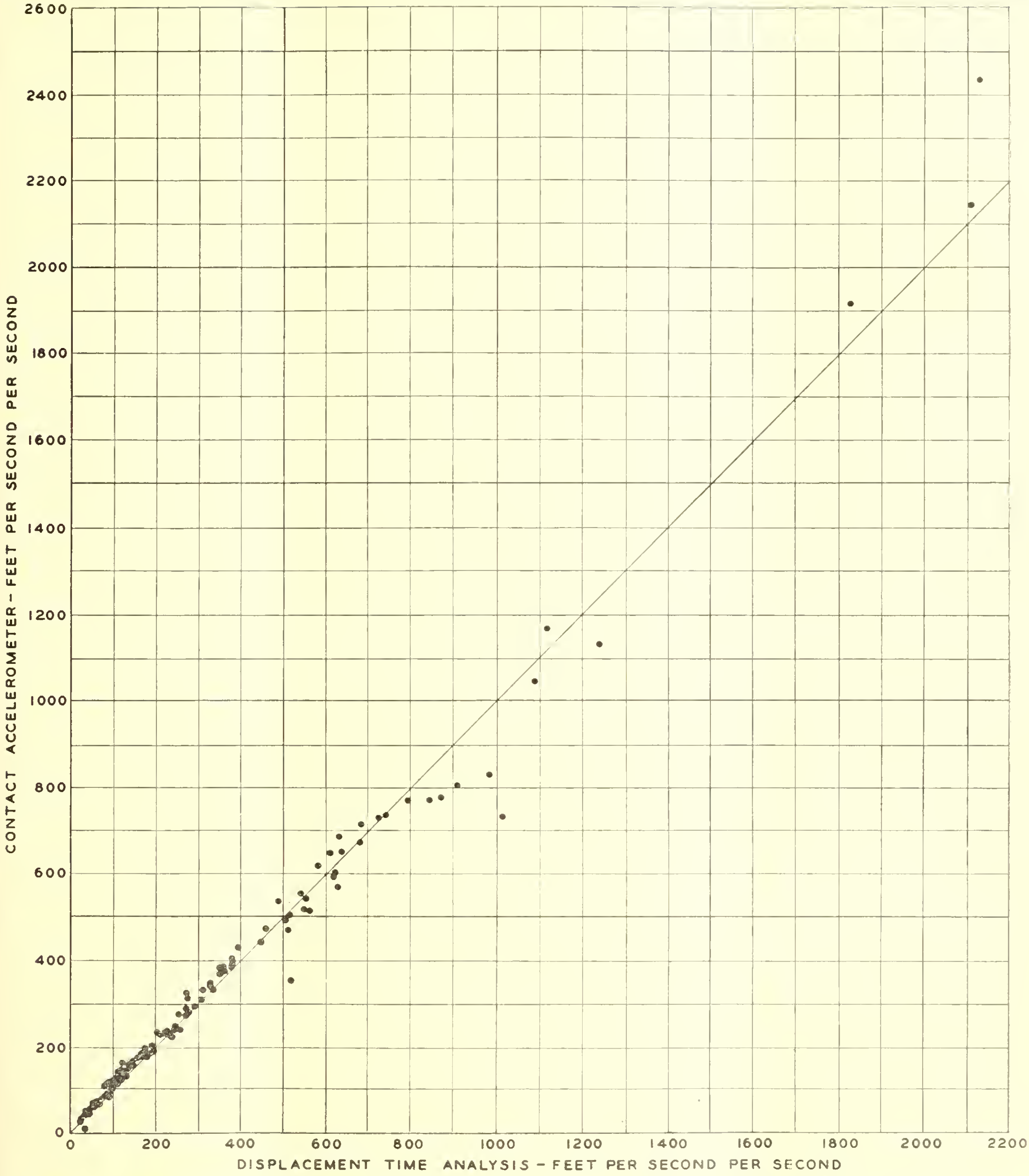


Figure 11.—Reaction Wheel Tests. Acceleration Indicated by Single-Element Contact Accelerometer Plotted Against Acceleration Computed by Analysis of Displacement-Time Curve

The coil spring accelerometer differs from other instruments involving spring-weight elements in the fact that values of n are generally low, lying in most cases between 1 and 3.5. Values less than 1 are found occasionally, but values greater than 5 seldom if ever occur in motor truck impact tests. The peculiar properties of the instrument depend upon this range in the values of n, which gives rise to a variation of the calibration factor with the period of the impressed motion.

For values of n lying between 0 and 5, with the exception of the value $n = 1$, the maximum deflection of the element is given by the equation,

$$R = \frac{a}{n(n-1)} \sin \frac{2\pi}{n+1}, \text{ Case I} \ldots\ldots (5)$$

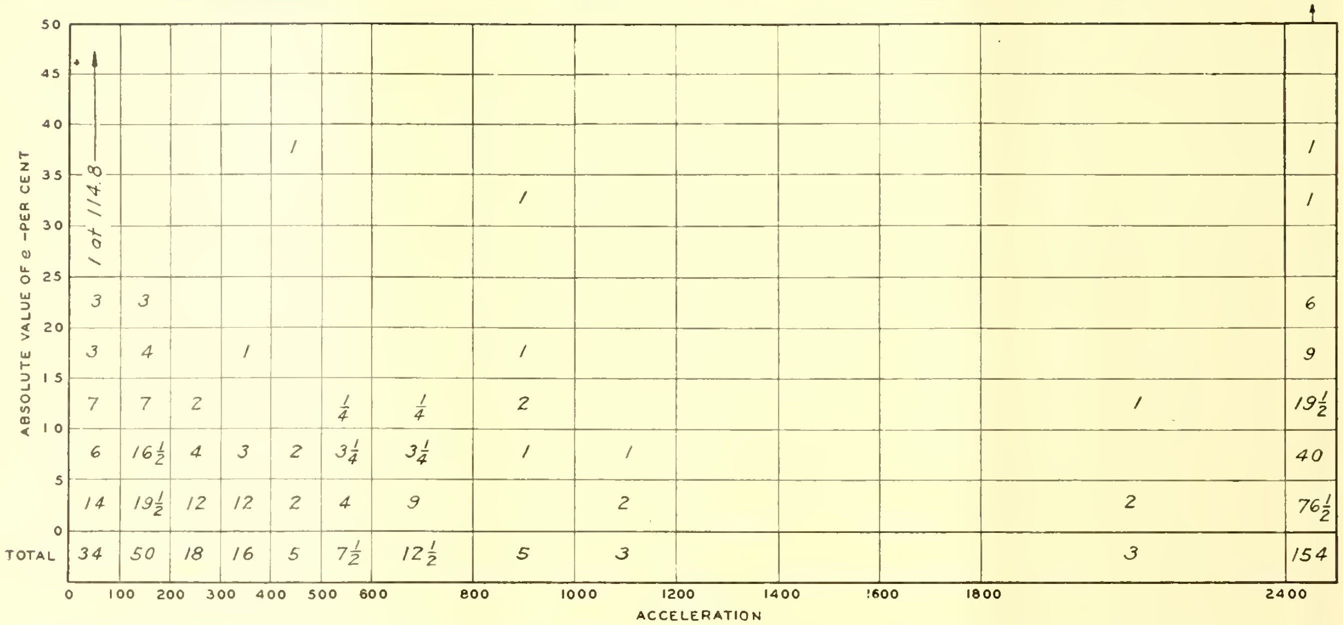


FIGURE 12.—FREQUENCY DISTRIBUTION CHART FOR THE RELATION BETWEEN VALUES OF $e=100\times\frac{A_{DT}-A_C}{A_M}$ AND A_M

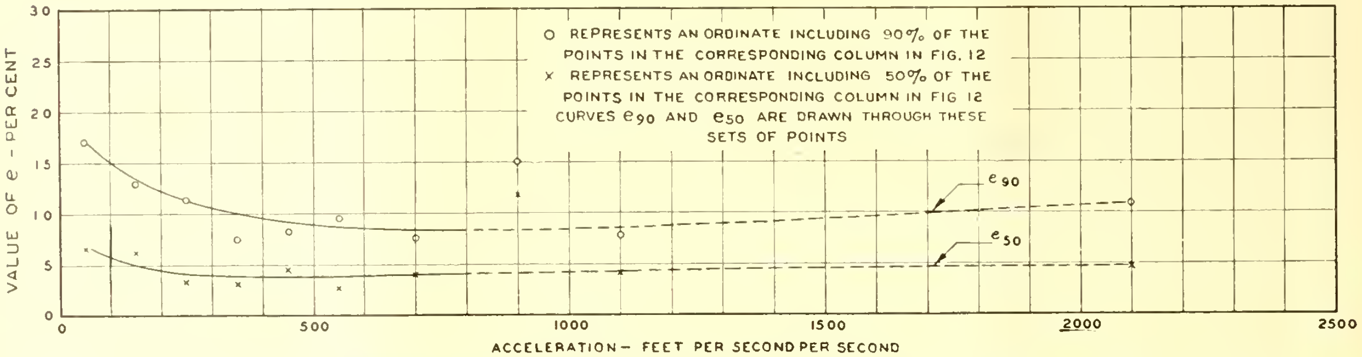


FIGURE 13.—CURVES SHOWING PROBABLE VARIATION BETWEEN ACCELERATIONS (A_C) GIVEN BY CONTACT ACCELEROMETER AND ACCELERATIONS (A_{DT}) GIVEN BY DISPLACEMENT-TIME ANALYSIS. (SEE FIG. 12)

When $n=1$,

$$R=\frac{\pi}{2}a,\ \text{Case II}\ \text{------------------}\ (6)$$

The calibration factor, C, is defined by the formula,

$$C=-\frac{A}{R}$$

where A = the maximum acceleration acting upon the frame (negative when R is positive).

In the case of simple harmonic motion,

$$A=\frac{-4\pi^2}{T_1^2}a,$$

and the equations for C take the forms,

$$C=\frac{n-1}{n}\frac{4\pi^2}{T_2^2}\operatorname{cosec}\frac{2\pi}{n+1}$$

$$=\frac{n-1}{n}\frac{gs}{w}\operatorname{cosec}\frac{2\pi}{n+1},\ \text{Case I}\ \text{------}\ (7)$$

$$C=\frac{4\pi^2}{T^2}\frac{2}{\pi}=\frac{gs}{w}\frac{2}{\pi},\ \text{Case II}\ \text{--------}\ (8)$$

The quantity $c=\frac{C}{\frac{gs}{w}}$ is defined as the relative calibration factor. Substituting this formula in equations (7) and (8) we have

$$c=\frac{n-1}{n}\operatorname{cosec}\frac{2\pi}{n+1},\ \text{Case I}\ \text{-------}\ (9)$$

$$c=\frac{2}{\pi}=0.6366,\ \text{Case II}\ \text{--------}\ (10)$$

THE *n* vs. *c* RELATION USED AS BASIS OF ANALYSIS

It is evident that the relative calibration factor, c, is a function of n alone. The relation between n and c is shown in Figure 16 for values of n from 0.4 to 5.0. The special case, $n=1$, $c=0.6366$, lies on this curve. In subsequent pages this function is referred to as the theoretical n vs. c relation, or n vs. c curve. A notable characteristic of this function is that within a wide range in values of n the value of c varies but little. For example, a value of 0.62 could be used for c with errors no greater than 10 per cent for all values of n between 0.87 and 3.20. This characteristic of the coil spring accelerometer is very important, as it makes

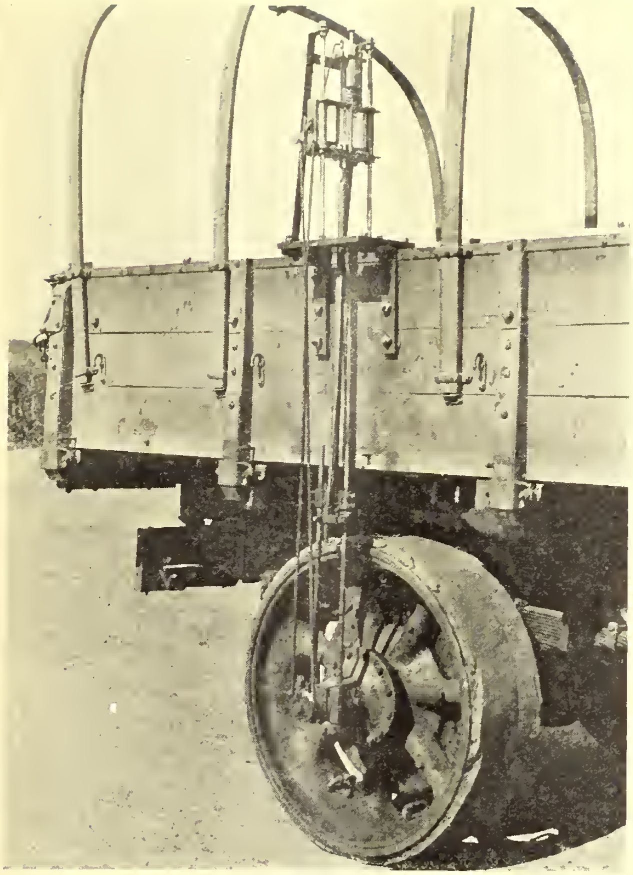


Figure 14.—A Test Truck Equipped for Road Tests With the Coil Spring Accelerometer

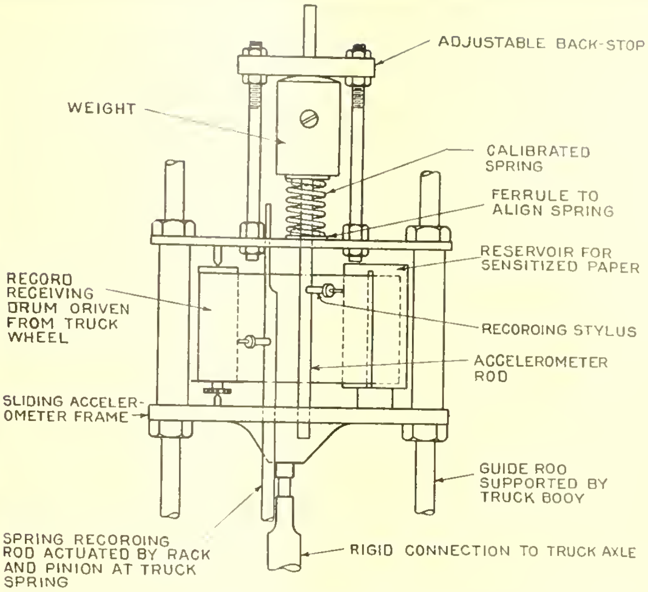

ADJUSTABLE BACK-STOP
WEIGHT
CALIBRATED SPRING
FERRULE TO ALIGN SPRING
RESERVOIR FOR SENSITIZED PAPER
RECORD RECEIVING DRUM DRIVEN FROM TRUCK WHEEL
RECORDING STYLUS
ACCELEROMETER ROD
SLIDING ACCELEROMETER FRAME
GUIDE ROD SUPPORTED BY TRUCK BODY
SPRING RECORDING ROD ACTUATED BY RACK AND PINION AT TRUCK SPRING
RIGID CONNECTION TO TRUCK AXLE

Figure 15.—Diagram of Essential Elements of Coil Spring Accelerometer

possible the use of a constant calibration factor without great error, even though the periods of the accelerations to be measured vary between fairly wide limits.

The development of the *n* vs. *c* relation opened two avenues of approach to the problem of the coil spring accelerometer. Since it is based upon simple harmonic motion, it made possible a direct check between the theory and the actual performance of the instrument in tests made on the simple harmonic machine at the Bureau of Standards. It also served as the basis for a

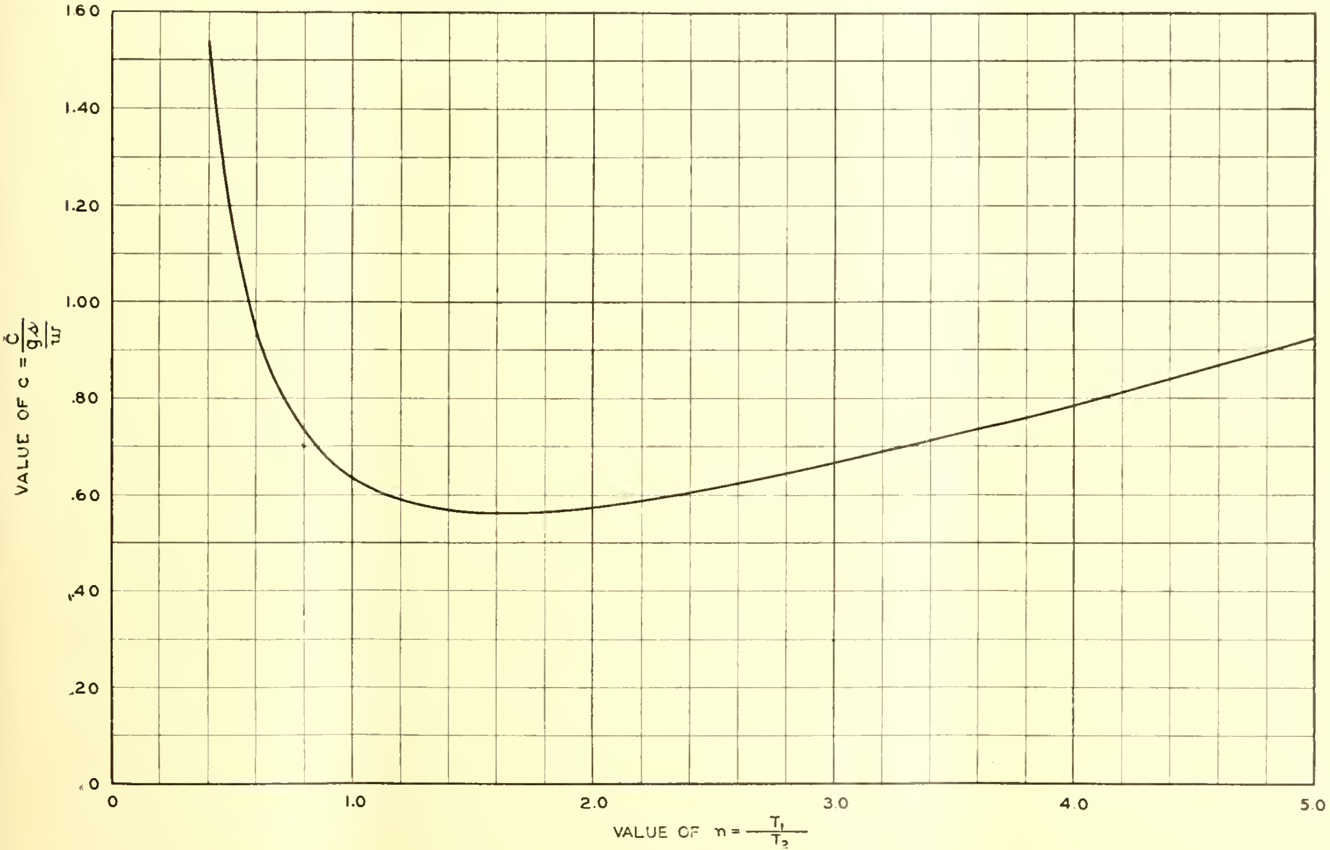


Figure 16.—Functional Relation Between the Ratios $c=\frac{C}{\frac{gs}{w}}$ and $n=\frac{T_1}{T_2}$ as Determined Theoretically

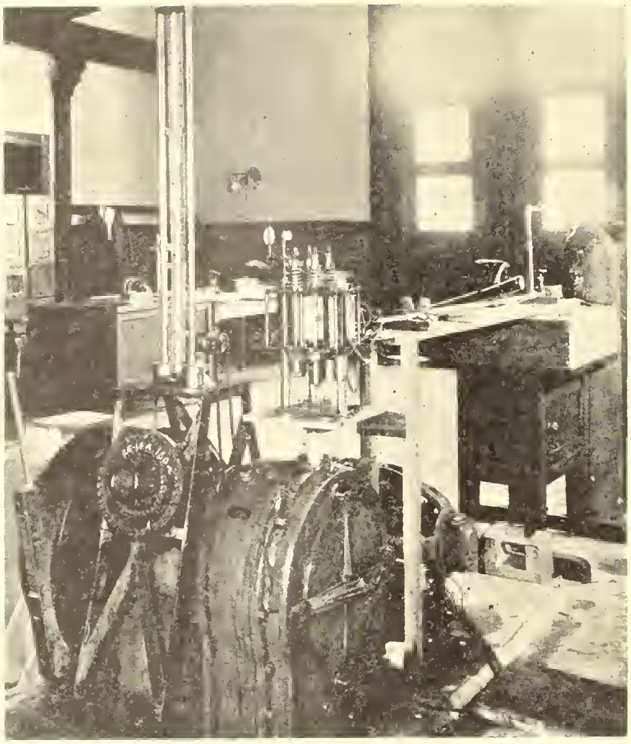


Figure 17.—Four-Element Coil Spring Accelerometer Installed on the Simple Harmonic Motion Machine

systematic analysis of the data obtained in the reaction wheel tests.

SPECIAL FOUR-ELEMENT COIL SPRING ACCELEROMETER CONSTRUCTED

For use in tests described in this report a special accelerometer carrying four spring-weight elements was constructed. By means of this instrument simultaneous records of four elements having different physical characteristics were obtained in each test. The quantity $\frac{gs}{w}$ was used as a criterion in the selection of springs. Values of s were determined by static calibration and values of w by weighing on a beam balance. A different group of elements was selected for each type of tire tested on the reaction wheel. The selection was made so as to include two springs which would normally be used with the given type of tire, one spring having a value of $\frac{gs}{w}$ lower than normal and one spring having a value higher than normal.

For convenience in operation on the reaction wheel and the simple harmonic motion machine, the recording styli were arranged so as to bear upon the record paper by the application of pneumatic pressure. Figure 17 shows the instrument mounted on the simple harmonic motion machine. Figure 2 shows its installation for use in the reaction wheel tests.

FOUR GROUPS OF TESTS MADE ON THE SIMPLE HARMONIC MOTION MACHINE

The tests conducted on the simple harmonic motion machine were designed to simulate, with respect to amplitude and period, the impacts obtained in tests with four different types of tire. For this reason the tests were divided into four groups and for each group the four spring-weight elements were selected which were used in the reaction wheel tests, the type of tire being simulated by the given group of simple harmonic tests. Table 7 gives the test conditions for each group, together with the spring-weight elements used and their characteristics. It will be observed that tests were run at speeds ranging from 299 to 2,020 revolutions per minute. These speeds correspond to periods of cycle ranging from 0.2007 to 0.0297 second and include all drop periods encountered in motor-truck impact tests except those obtained on tests with badly worn solid tires.

For each test condition a series of autographic records was obtained from each of the four accelerometer elements. The lengths of these records were measured, and the mean of the series of records for each element at a given test condition, designated as R_m, was taken as the basis for the computations.

Table 7.—*Tests with the coil spring accelerometer on the simple harmonic motion machine*

Tire type simulated	Test conditions		Accelerometer elements		
	Stroke	Revolutions per minute	No.	$\frac{gs}{w}$	T_2
	Inches			*Feet per second per second per inch*	*Seconds*
Pneumatic			51A	1,653	0.0446
	1.006	299 to 599	102B	2,784	.0344
	1.499	299 to 599	52C	908	.0602
	1.992	299 to 599	53D	588	.0748
Cushion			51A	1,653	.0446
	1.006	299 to 932	102B	2,784	.0344
	1.499	599 to 882	52C	908	.0602
	1.992	299 to 799	401D	4,497	.0270
Solid			51A	1,653	.0446
	0.500	745 to 1,570	102B	2,784	.0344
	1.000	745 to 1,420	401C	6,718	.0221
			402D	4,526	.0270
Worn solid			401A	11,606	.0168
	0.500	1,345 to 2,020	601B	15,841	.0144
			402C	6,760	.0221
			201D	2,401	.0370

EMPIRICAL *n* vs. *c* RELATION FOR SIMPLE HARMONIC MOTION DEVELOPED

In order to obtain an empirical function analogous to the n vs. c relation derived theoretically, values of n and c were computed from the formulas $n=\frac{T_1}{T_2}$ and $c=-\frac{A}{R_m}\div\frac{gs}{w}$. For each test there are, in general, four values of R_m, and therefore four sets of n vs. c points, or coordinates. These points, 214 in number, were plotted and the plotting was used as the basis of the frequency distribution diagram of Figure 18. The figure within a given square indicates the number of points lying within the same coordinate limits on the original plot of n vs. c coordinates. A point lying on a boundary line was divided equally between the adjacent squares. It will be observed that there are a number of points for which the value of n is greater than 5. Such values do not occur in motor-truck impact and for that reason these points are neglected in the subsequent portion of the discussion.

Figure 19 shows the method followed in obtaining an average curve to represent the empirical n vs. c relation for simple harmonic motion. In this figure are plotted the median value and the arithmetical mean value of c for each column of squares in Figure 18. Little distinction is shown between the median and the

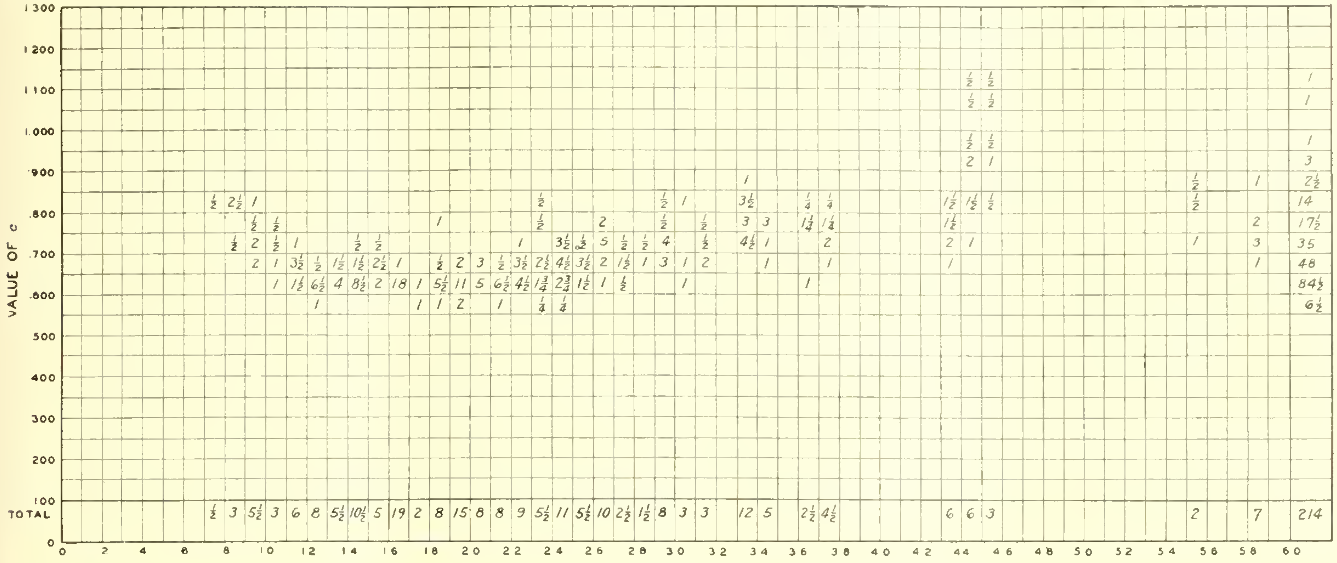


FIGURE 18.—FREQUENCY DISTRIBUTION CHART FOR THE n VS. c RELATION INDICATED BY TESTS ON THE SIMPLE HARMONIC MOTION MACHINE. THE NUMBER WITHIN EACH SQUARE INDICATES THE NUMBER OF POINTS FALLING WITHIN ITS COORDINATE LIMITS

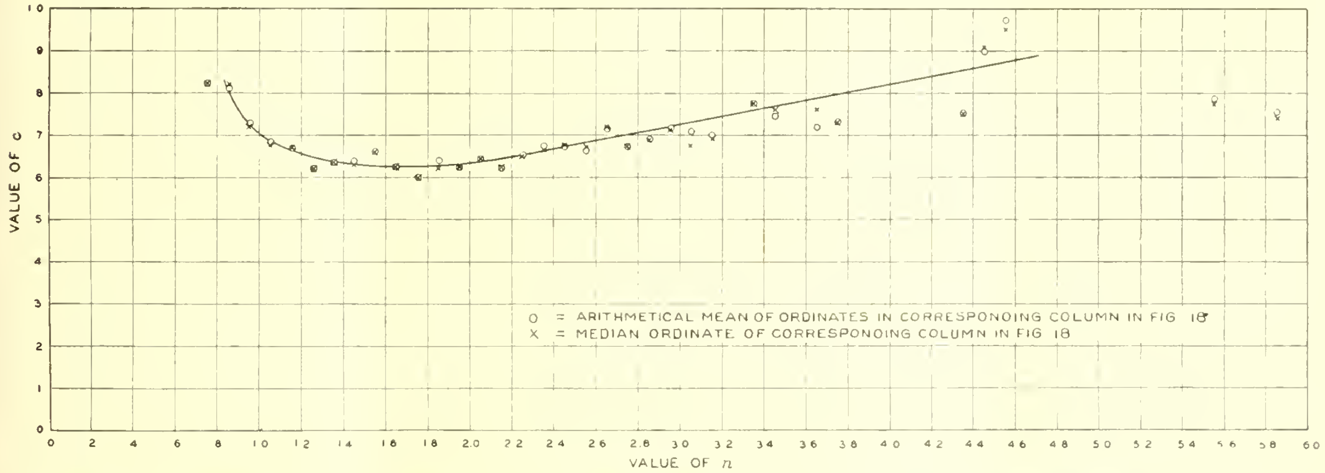


FIGURE 19.—DEVELOPMENT OF AVERAGE n VS. c CURVE FROM TESTS ON THE SIMPLE HARMONIC MOTION MACHINE

mean, although the former tends to reduce the effect of the more widely scattered points. The average curve (fig. 19) was drawn through these points, weight being given according to the frequency represented by each point.

From an inspection of Figures 18, 19, and 16, the following conclusions may be drawn:

(1) There is a noticeable dispersion in the values of c.

(2) The empirical relation is similar to the theoretical relation, with respect to both magnitude and manner of variation, even for high values of n, where the dispersion is greatest.

(3) Within the proper working range of the instrument (from $n=1.0$ to $n=3.5$) the points tend to lie above the theoretical curve. For high values of n they tend to lie below it.

DISPERSION OF DATA STUDIED

To serve as the basis for a study of the dispersion of the data presented in Figure 18, the quantity, e, is defined by the formula $e=100\left(\frac{c}{c'}-1\right)$, where

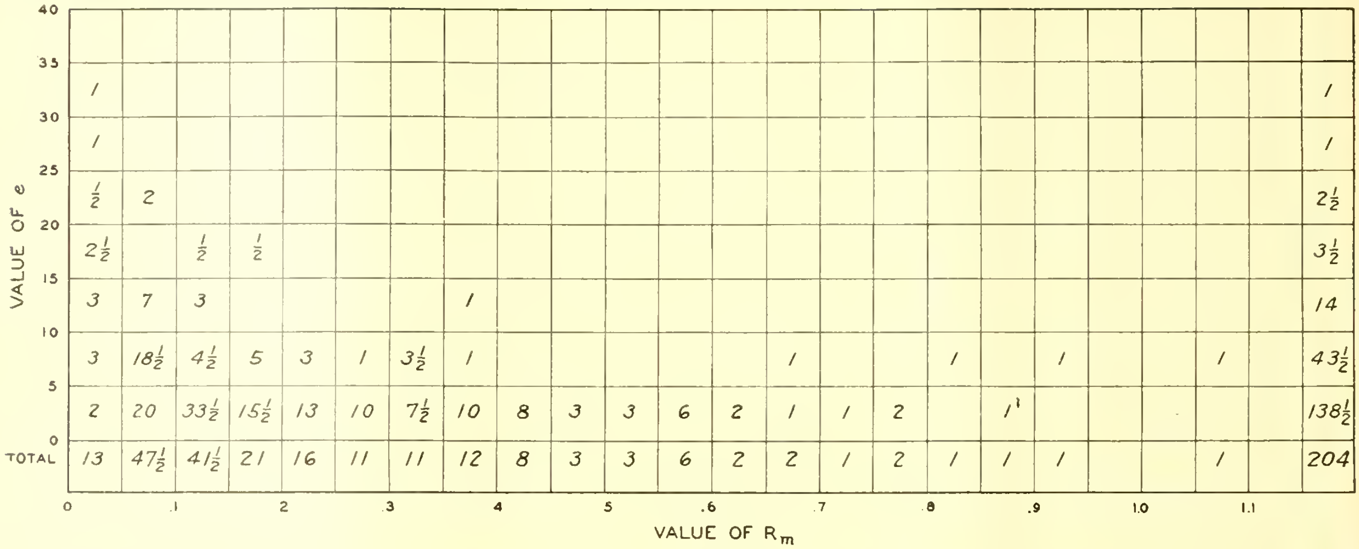
c = the experimental value of c obtained from a given test.

c' = the value of c obtained from the average curve for the given value of n.

The quantity e, therefore, may be defined as the percentage deviation of any given experimental value of c from the average value given by the curve.

Values of e were computed for all the experimental points except those lying beyond the limits of the average curve (fig. 19). In all, there are 204 such values. The results of this computation are shown in Figure 20 in the form of a frequency pattern, showing the distribution of values of e, without distinction of sign, against variation in length of record. There is a clear indication that the dispersion decreases as the length of record increases.

For each column of squares in Figure 20, ordinates were chosen to include 50 per cent and 90 per cent of the points, respectively. The resulting data are shown plotted in Figure 21, and the curves e_{50} and e_{90} are drawn through them. The significance of the curve e_{50} may be stated as follows: If the value of n is known the corresponding value of c' may be used in computing



1																							1
1																							1
½	2																						2½
2½		½	½																				3½
3	7	3					1																14
3	18½	4½	5	3	1	3½	1						1			1		1			1		43½
2	20	33½	15½	13	10	7½	10	8	3	3	6	2	1	1	2		1						138½
13	47½	41½	21	16	11	11	12	8	3	3	6	2	2	1	2	1	1	1			1		204

Figure 20.—Frequency Distribution Chart for the Absolute Values of the Error, e, With Respect to the Accelerometer Reading, R_m, for Tests on the Simple Harmonic Motion Machine. The Number Within Each Square Indicates the Number of Points Falling Within its Coordinate Limits

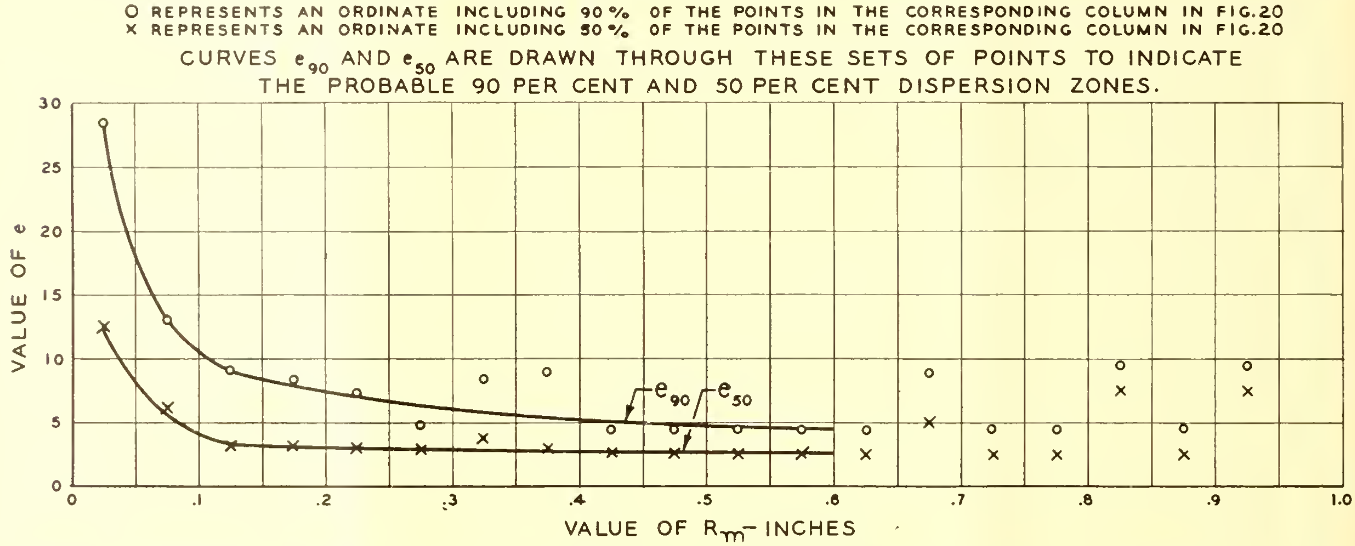


Figure 21.—Relation Between Accelerometer Error, e, and Accelerometer Reading, R_m, Indicated by the tests on the Simple Harmonic Motion Machine. (See Fig. 20.)

accelerations by means of the formula, $-A=c'\frac{gs}{w}R$, with an even chance, or 0.5 probability, that the true acceleration lies within a percentage of the acceleration so computed, equal to the ordinate on the e_{50} curve corresponding to the value of R obtained in the test for which the computation is made. There is, similarly, a 9 to 1 chance, or 0.9 probability, that the true acceleration lies within a percentage of the computed value equal to the corresponding ordinate on the curve e_{90}. These statements, naturally, apply only to the case of simple harmonic motion.

THE REACTION WHEEL TESTS DISCUSSED

The method used in analyzing the data obtained from the reaction wheel tests is closely patterned after that used in the case of simple harmonic motion. This procedure was adopted because it was believed that, in motor truck impact as in simple harmonic motion, the duration and the intensity of the impressed acceleration are the chief factors determining the length of the coil spring acceleromotor record. If this were not so it would be practically impossible to achieve a systematic calibration of the instrument. The addition of other variables to the system would complicate the problem beyond all hope of a satisfactory solution. It is recognized, however, that other factors must necessarily influence the deflection of the spring to some degree. The displacement-time curves, although sinusoid in character, are not simple sine curves nor can they, as a group, be represented by any other simple function. If we should assume that they may be represented by a large number of functions varying in different degrees from the sine we would expect to obtain an equal number of n vs. c relations. These taken together would form a band, the width of the band and the distribution of the curve within it indicating the extent to which the functions differed among themselves. From this consideration alone we should anticipate a dispersion band of some width in the n vs. c relation exhibited by the reaction wheel data. In addition, we have the natural dispersion of the instrument as shown by the tests on the simple harmonic motion machine.

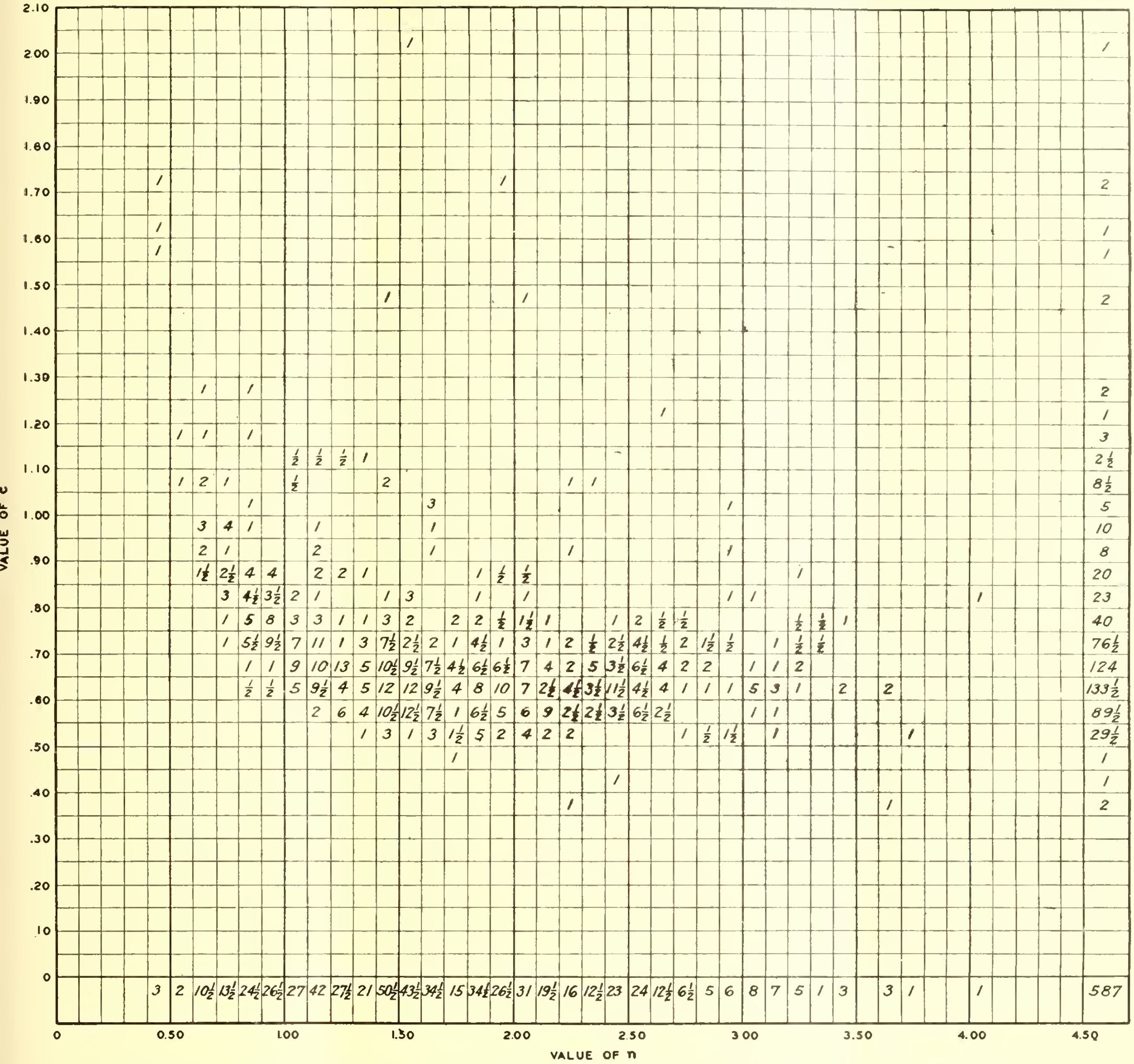


FIGURE 22.—FREQUENCY DISTRIBUTION CHART FOR THE n VS. c RELATION INDICATED BY THE REACTION WHEEL TESTS. THE NUMBER WITHIN EACH SQUARE INDICATES THE NUMBER OF POINTS FALLING WITHIN ITS COORDINATE LIMITS

EMPIRICAL n vs. c RELATION DEVELOPED FOR REACTION WHEEL DATA

In this analysis the same quantities are involved as in the case of simple harmonic motion with certain changes in definition. We have:

T_1 = twice the period of duration of the impact as measured on the displacement-time curve.

T_2 = free period of vibration of the accelerometer element.

$$n=\frac{T_1}{T_2}.$$

A = The maximum acceleration of the impact. The value used is the mean of the values given by the contact accelerometer and the displacement-time recorder.

R = length of the accelerometer record.

$$C=\frac{-A}{R}.$$

$$c=C\div\frac{gs}{w}.$$

For each test there are four values of R, and therefore four sets of n vs. c coordinates except in a few cases where one or more elements failed to record properly. Five hundred and eighty-seven such coordinates were obtained from the computations, and their distribution is shown in the frequency pattern of Figure 22. An examination of this frequency distribution reveals the following facts.

1. There is a rather wide dispersion of the points.
2. Despite this dispersion, a systematic variation of c as a function of n is strongly indicated. The mean position of the points is displaced so as to lie in general somewhat above the theoretical curve, but is of similar shape.
3. The dispersion tends to be normal, i. e., as the center of the dispersion band is approached the frequency of the points increases rapidly.

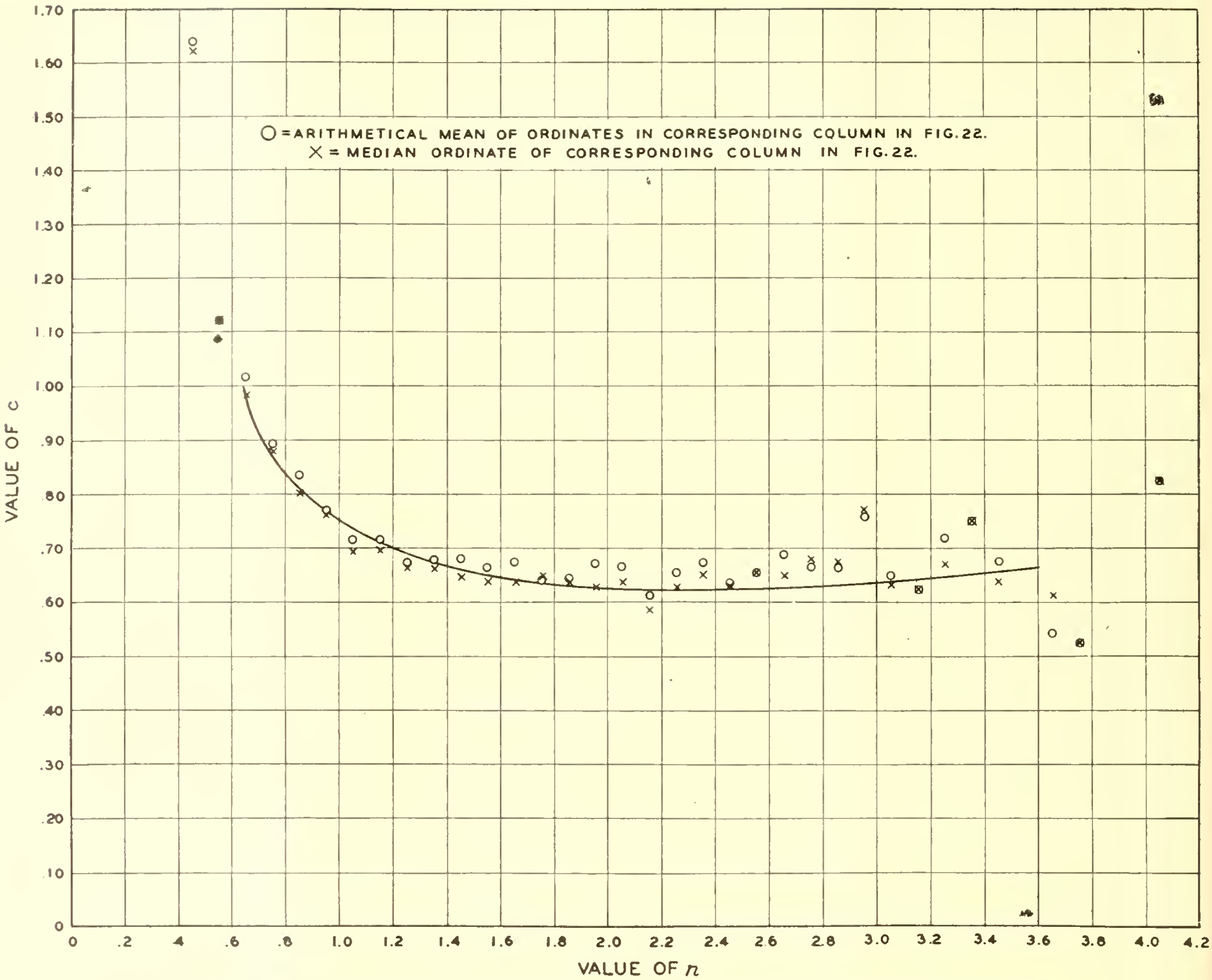


Figure 23.—Development of Average n vs. c Curve for Reaction Wheel Data

4. There is some scattering of points at a great distance from the mean position. It is evident that such points represent conditions highly unfavorable to the use of the coil spring accelerometer.

In Figure 23 the median value and the arithmetical mean value of c are plotted for each column of squares in Figure 22. As in the case of the tests on the simple harmonic motion machine, a mean curve has been drawn. Greater weight was given to the median than to the mean in drawing this curve in order to discount the effect of widely scattered points. In subsequent pages of this report this curve is used as the basic n vs. c relation for motor-truck impact. In Figure 24 the three n vs. c relations derived in this investigation are given together for the purposes of intercomparison.

RELATION FOUND BETWEEN DISPERSION OF REACTION WHEEL DATA AND LENGTH OF ACCELEROMETER RECORD

Upon examination of the reaction wheel data it was found that nearly all the widely dispersed points in the frequency pattern of Figure 22 correspond to accelerometer records of small length, a great many of them measuring less than 0.05 inch. The same procedure was adopted as in the case of tests on the simple harmonic motion machine. Values of c taken from the average curve of Figure 23 are given the symbol c'. The quantity $e = 100\left(\frac{c}{c'} - 1\right)$ is defined as the percentage deviation of any given experimental value of c from the average value c' for the corresponding value of n. Values of e were computed for all experimental points except those for which the values of n lie outside the limits of the average n vs. c curve. The distribution of the values of e, 568 in all, is shown in the frequency pattern of Figure 25. As in Figure 20, absolute values of e are given. This diagram indicates very clearly that the dispersion increases rapidly as the length of record approaches zero. The tendency toward concentration near the zero line persists, even for very low values of R. It is plain, however, that results obtained from records less than 0.1 inch in length are unreliable. This fact does not seriously affect the value of the coil spring accelerometer. If the spring-weight element is properly chosen such extremely short records will be obtained only when the accelerations are very low and therefore unimportant.

As in the case of the tests on the simple harmonic motion machine, ordinates were determined including 50 per cent and 90 per cent of the values of e in each column of squares in Figure 25. These ordinates and the corresponding curves, e_{50} and e_{90}, are shown in

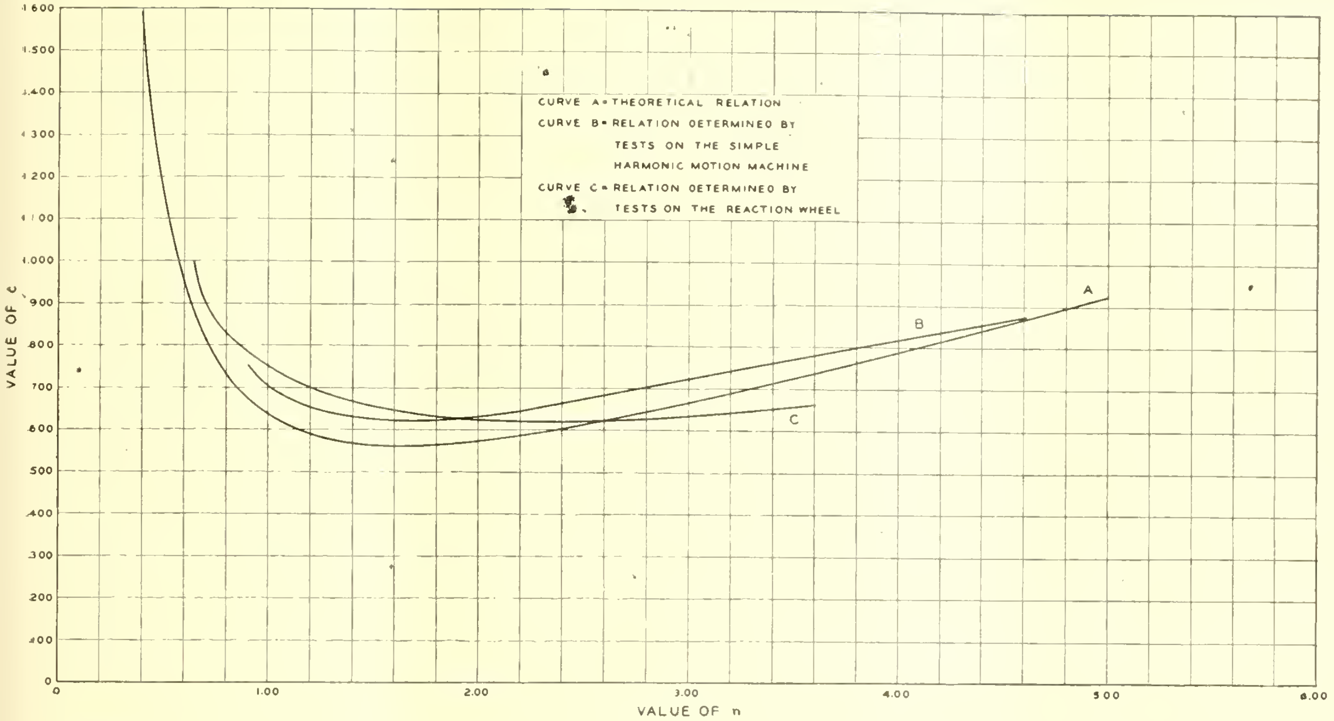


FIGURE 24.—THE CALIBRATION COEFFICIENT c, IN THE EQUATION $C=c\frac{gs}{w}$, AS A FUNCTION OF THE RATIO $n=\frac{T_1}{T_2}$

VALUE OF e \ VALUE OF R	0–.05	.05–.1	.1–.15	.15–.2	.2–.25	.25–.3	.3–.35	.35–.4	.4–.45	.45–.5	.5–.55	.55–.6	.6–.65	.65–.7	.7–.75	.75–.8	.8–.85	.85–.9	.9–.95	.95–1.0	1.0–1.05	1.05–1.1	
above 75	5																						5
70–75	2																						2
65–70	1	1																					2
60–65	1	1																					2
55–60	2	2																					4
50–55	1	2																					3
45–50	1		1																				2
40–45	2	1					1																4
35–40	2	3	1																				6
30–35	1	1	1																				3
25–30	4	4	1																				9
20–25	3	7	6	2		1	1																20
15–20	5	16½	14½	6½	3			1			2				1								49½
10–15	12	29	22	15½	10	2½	5	4½	4			2	1	2	1	1							111½
5–10	17½	29½	30	10	19	10	6	4½	5½	4		1½		1									138½
0–5	22½	44	36	19½	26	14	15½	9½	3	5	1	7½				1					1	1	206½
TOTAL	82	141	112½	53½	58	27½	28½	19½	12½	9	3	11	1	3	2	2					1	1	568

IGURE 25.—FREQUENCY DISTRIBUTION CHART FOR THE ABSOLUTE VALUES OF THE ERROR, e, WITH RESPECT TO ACCELEROMETER READING, R, AS GIVEN BY THE REACTION WHEEL TESTS. THE NUMBER WITHIN EACH SQUARE INDICATES THE NUMBER OF POINTS FALLING WITHIN ITS COORDINATE LIMITS

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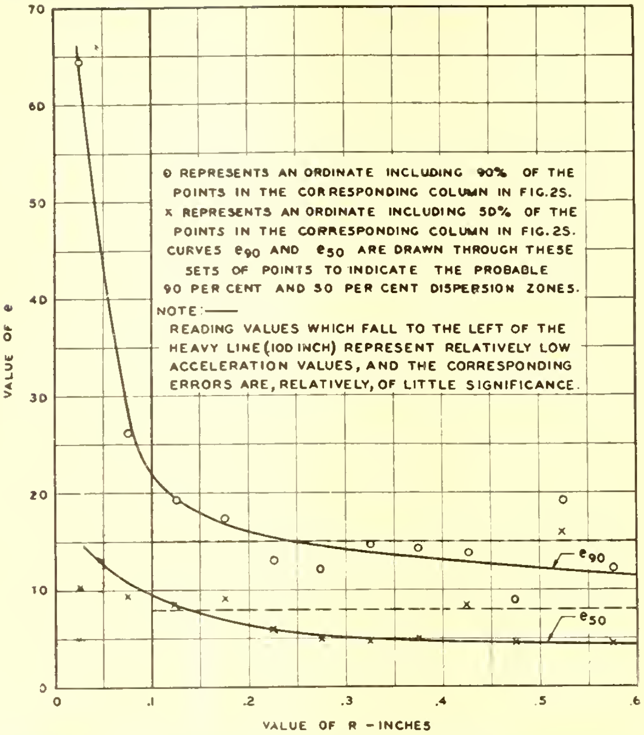

FIGURE 26.—RELATION BETWEEN ACCELEROMETER ERROR, e, AND ACCELEROMETER READING, R, INDICATED BY THE REACTION WHEEL TESTS. (SEE FIG. 25.)

Figure 26. We may again define the significance of these curves, as applied to motor-truck impact tests, by the following statements. If, in any impact test, the value of n is known with sufficient accuracy for use with the n vs. c' curve of Figure 23, the corresponding value of c' may be used in the formula,

$$-A = c'\frac{gs}{w}R,$$

with an even chance, or 0.5 probability, that the true acceleration lies within a percentage of the acceleration so computed equal to the ordinate on the curve e_{50} corresponding to the value of R obtained in the test. In the case of the curve e_{90} there is a 9 to 1 chance, or 0.9 probability, that the true acceleration lies within a percentage of the computed value equal to the corresponding ordinate on that curve. Although the procedure adopted in obtaining these functions is not rigorous, it is believed that the curves e_{50} and e_{90} are fair indices of the type of dispersion to be expected when the n vs. c' relation of Figure 23 is used as a basis for computing motor truck impact accelerations from records of the coil spring accelerometer.

It will be noted that the curves e_{50} and e_{90} are drawn through the bulk of the points on Figure 26. These curves were drawn to express the general relation between dispersion and the length of accelerometer record and are used in subsequent computations. It will be observed that, for each of these curves, several points representing frequencies which should not be neglected lie definitely above the curve as drawn. For the purpose of making general deductions, two additional curves have been drawn in broken line, at values of e equal to 8 and 15 per cent, which, within the significant range of record lengths, definitely include more than 50 per cent and more than 90 per cent of the data, respectively.

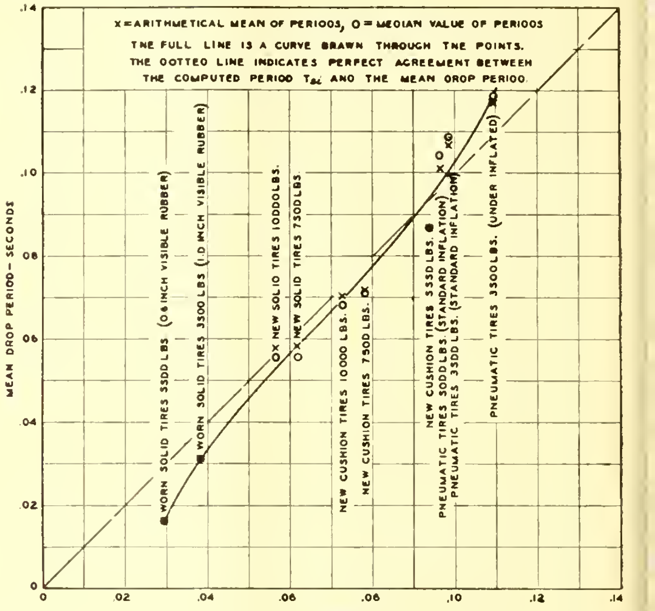

FIGURE 27.—GRAPH OF MEAN DROP PERIOD AS A FUNCTION OF T_{si}

THE USE OF THE n VS. c RELATION IN IMPACT COMPUTATIONS DISCUSSED

The empirical n vs. c' relation developed in the preceding discussion and illustrated in Figure 23 may be regarded as the basis of the "most probable" calibration system for the coil spring accelerometer. In order to make use of this relation it is necessary to determine the period of impact T_1 for use in computing the ratio $n=\frac{T_1}{T_2}$. It is not required that the period be known with great accuracy, since the relative calibration factor c' varies very slowly within a wide range in values of n. Except for very low or very high values of n, an error of 10 or even 20 per cent in T_1 would not produce a large error in the value of c'. The probable error due to the dispersion of the instrument under impact conditions is the chief limitation upon the accuracy which may be expected from the use of the n vs. c' relation.

In order to find a method for estimating the period of impact from known test conditions it is necessary to determine the relations existing between that period and those variables (such as load, speed, unsprung weight, type of obstruction and type of tire) which may be supposed to influence its magnitude. In the reaction wheel tests shock and drop periods were measured for a wide variety of test conditions. These data were analyzed with respect to the variables listed above and methods of estimating the value of T_1 were devised which give values in good agreement with the shock and drop periods obtained in the reaction wheel tests.

DROP PERIOD COMPUTED BY SIMPLE PROCEDURE

The analysis of drop periods presented a relativel simple problem. It was found that the period of dro is virtually independent of truck speed and height o obstruction, with a slight variation due to load. Th chief sources of variation proved to be tire type an unsprung weight. A relation was found to exist be tween the drop period T_1 and a quantity T_{si}, defined a the instantaneous period at static load. T_{si} is given b the formula $T_{si}=2\pi\sqrt{\frac{W}{gS_d}}$, where W= the unsprun weight and S_d = the loading rate of the tire or the tangent to the load-deflection curve at static load. This relation is shown in Figure 27, where mean drop periods are plotted against the corresponding values of T_{si}, computed from the known unsprung weight and the value of S_d obtained from the load-deflection curve of the tire. Each condition of tire and load used in the reaction wheel tests is represented by a circle and a cross, as indicated in the figure. The ordinate of each circle is equal to the median value of the drop periods for the given tire-load combination; the ordinate of each cross is equal to the arithmetical mean of the periods. A curve has been drawn to express the relation between mean drop period and T_{si} indicated by the points. It will be observed that the value of T_{si} could be used for T_I without great error for all tires except the worn solids. It is believed, however, that the curve drawn in Figure 27 offers a more reliable method of estimating the period of drop impact when the wheel load, the unsprung weight, and the load-deflection characteristics of the tire are known.

FUNCTION DEVELOPED FOR DETERMINING SHOCK PERIOD

The data regarding the period of shock proved more difficult of analysis. It was found necessary to go more deeply into the mechanics of the problem than in the case of drop to obtain a function expressing logically the manner in which the controlling test conditions influence the duration of the shock reaction. From an examination of the data the following facts were learned regarding the variation of shock period:

(1) It decreases as the truck speed increases.

(2) It decreases with the stiffness of the tire used.

(3) It increases with the height of obstruction.

(4) The effect of load is practically negligible, although there is some indication that the period varies inversely with the load.

In order to account for these sources of variation an equation of motion for the shock reaction was developed. This equation is identical in form with that derived for the motion of the accelerometer (equation 3). With altered coefficients the equation takes the form

$$y=\frac{ap^2}{q^2-p^2}\left(\sin pt-\frac{p}{q}\sin qt\right)\text{---------}(11)$$

Two types of shock reaction are to be considered: Case I, that occurring in reaction wheel tests; and Case II, that occurring in tests on a road surface. These two conditions are illustrated in Figures 28 and 29, respectively. The significance of the variables and coefficients in equation 11 is made plain by these figures and by the following list of definitions:

y = vertical displacement of the truck wheel at time t, measured from the point when upward motion begins.

In Figures 28 and 29—

R_2 = effective radius of the truck wheel, taken as equal to the overall radius of the tire, less its static deflection.

l = width of obstruction. Rectangular obstructions only are considered.

h = height of obstruction.

ψ = angle between a vertical line through the center of the truck wheel and a radius drawn to the edge of the obstruction at the instant of contact.

In Figure 28—

R_1 = radius of the reaction wheel.

α = the angle about the center of the reaction wheel subtended by the obstruction,

$$=\frac{l}{R_1+h}$$

β = the angle O_2O_1P. The value of β is obtained by solution of the triangle O_1PO_2, of which the three sides are known.

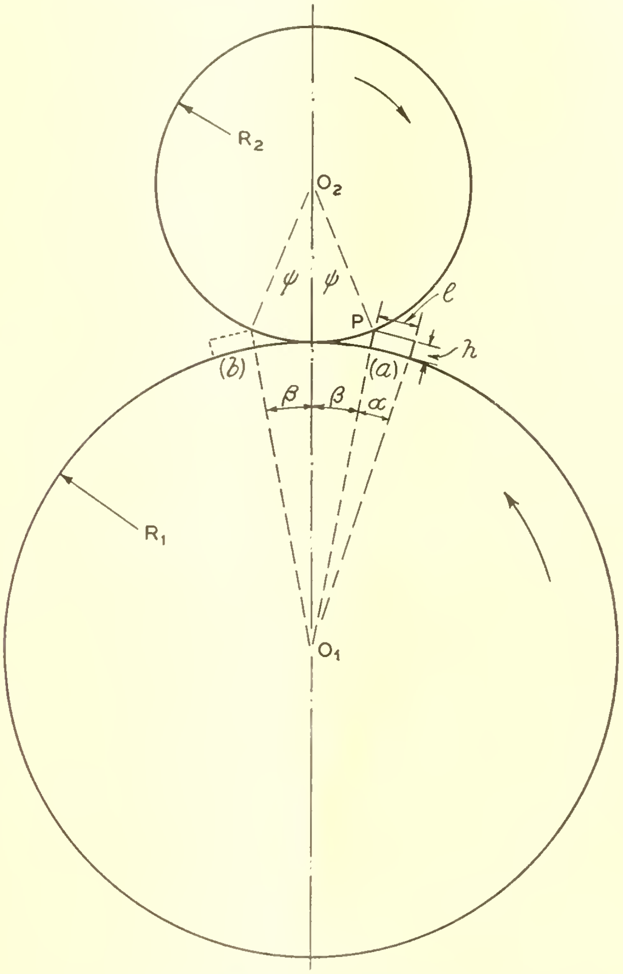

Figure 28.—Diagram of Tire Encountering Obstruction on Reaction Wheel

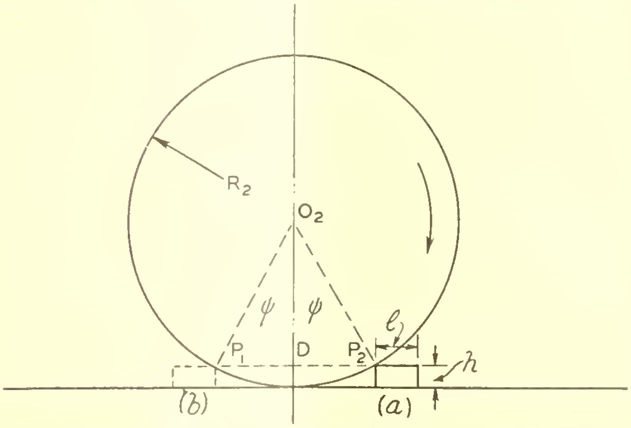

Figure 29.—Diagram of Tire Encountering Obstruction on Road Surface

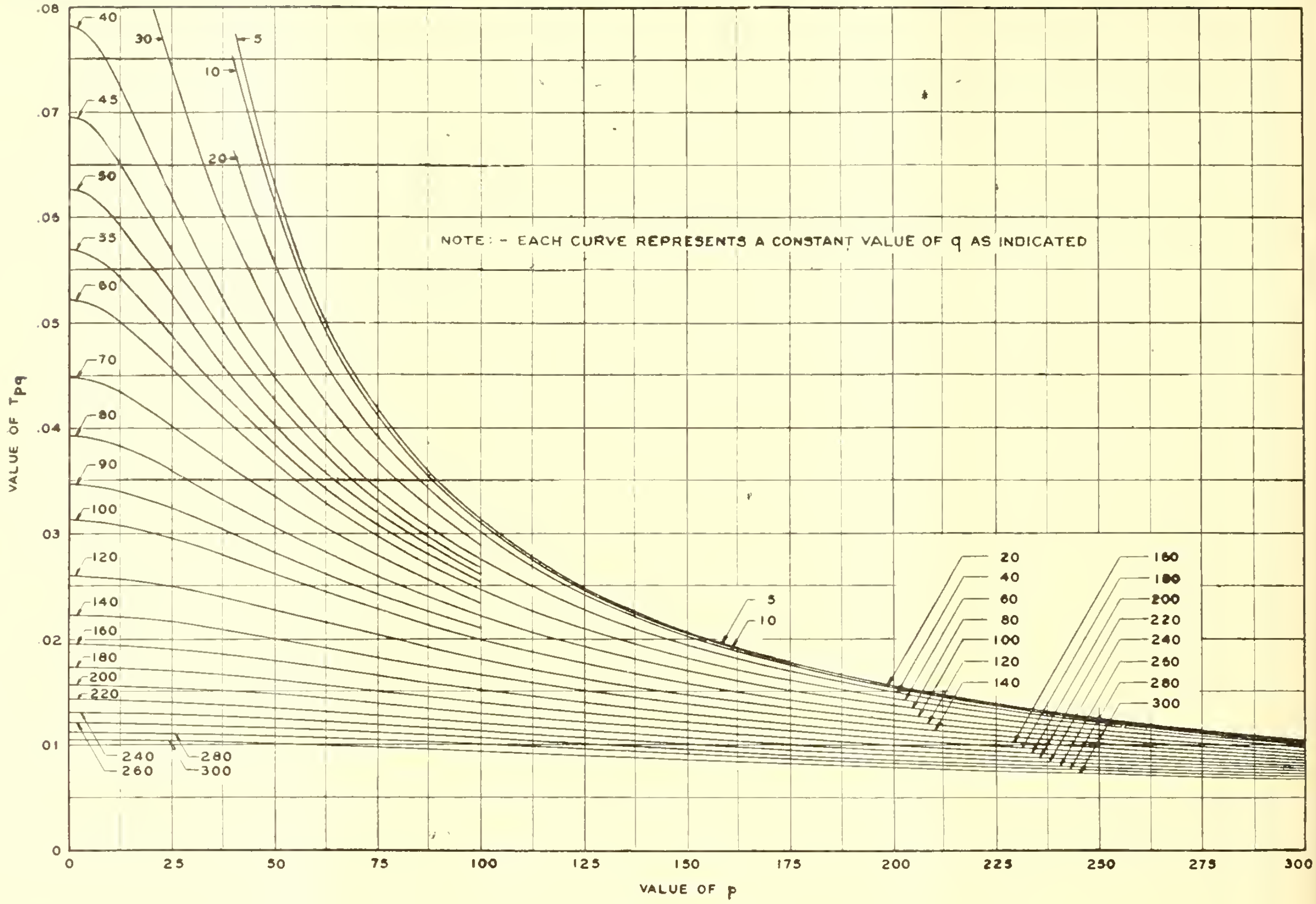

FIGURE 30.—GRAPH OF T_{pq} AS A FUNCTION OF p AND q. $p \sin pT_{pq}=q \sin qT_{pq}$

For Case I, on the reaction wheel,

$$p=\frac{\pi}{2}\,\frac{1}{\xi}\,\frac{V_t}{R_1}$$

where

V_t = "truck speed," i. e., tangential speed of truck wheel and reaction wheel.

$\xi=\beta+\frac{1}{2}\alpha$

For Case II, on the road surface,

$$p=\frac{\pi V_t}{l+2\sqrt{2R_2h-h^2}},$$

where V_t = truck speed.

For both Case I and Case II,

$$q=\sqrt{\frac{K+S}{m}}$$

$$a=\frac{Kh}{mp^2}$$

where

S = loading rate of truck spring,

m = unsprung mass,

K = a coefficient characteristic of the tire.

It will be observed that all the terms defined above with the exception of the coefficient K are definitely fixed by the test conditions. The problem of analysis consisted in finding an approximate or average value of K for each of the tires used in the reaction wheel tests and determining the relation between the values of K so obtained and the known static characteristics of the tires.

The shock impact ends at the first point of inflection of the displacement-time curve (see fig. 5, arc AB), i. e., at the instant upward acceleration ceases. The relation between p, q, and t at this instant is expressed by the following equation, derived from equation 11,

$$p \sin pt=q \sin qt \qquad (12)$$

This equation can not be solved for t by rational methods. A graphical solution was obtained, however, by means of a system of curves giving the half-period[4] of shock, $\frac{T_1}{2}$, as a function of p and q. This system of solutions is shown in Figure 30. Since the coefficient K appears in the formula for q but not in the formula for p, all values of the shock period for a given tire-truck combination should be satisfied by an equation of the form of equation 12, the value of q being constant. On this basis values of $\frac{T_1}{2}$ were plotted against values of p computed from the known test conditions, and superimposed upon the sustem of curves in Figure 30. By this means an average value of q was obtained for each tire-truck combination used in the reaction wheel tests, and the corresponding values of K were computed from the equation

$$q=\sqrt{\frac{K+S}{m}}$$

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[4] The actual duration of shock impact, as described above.

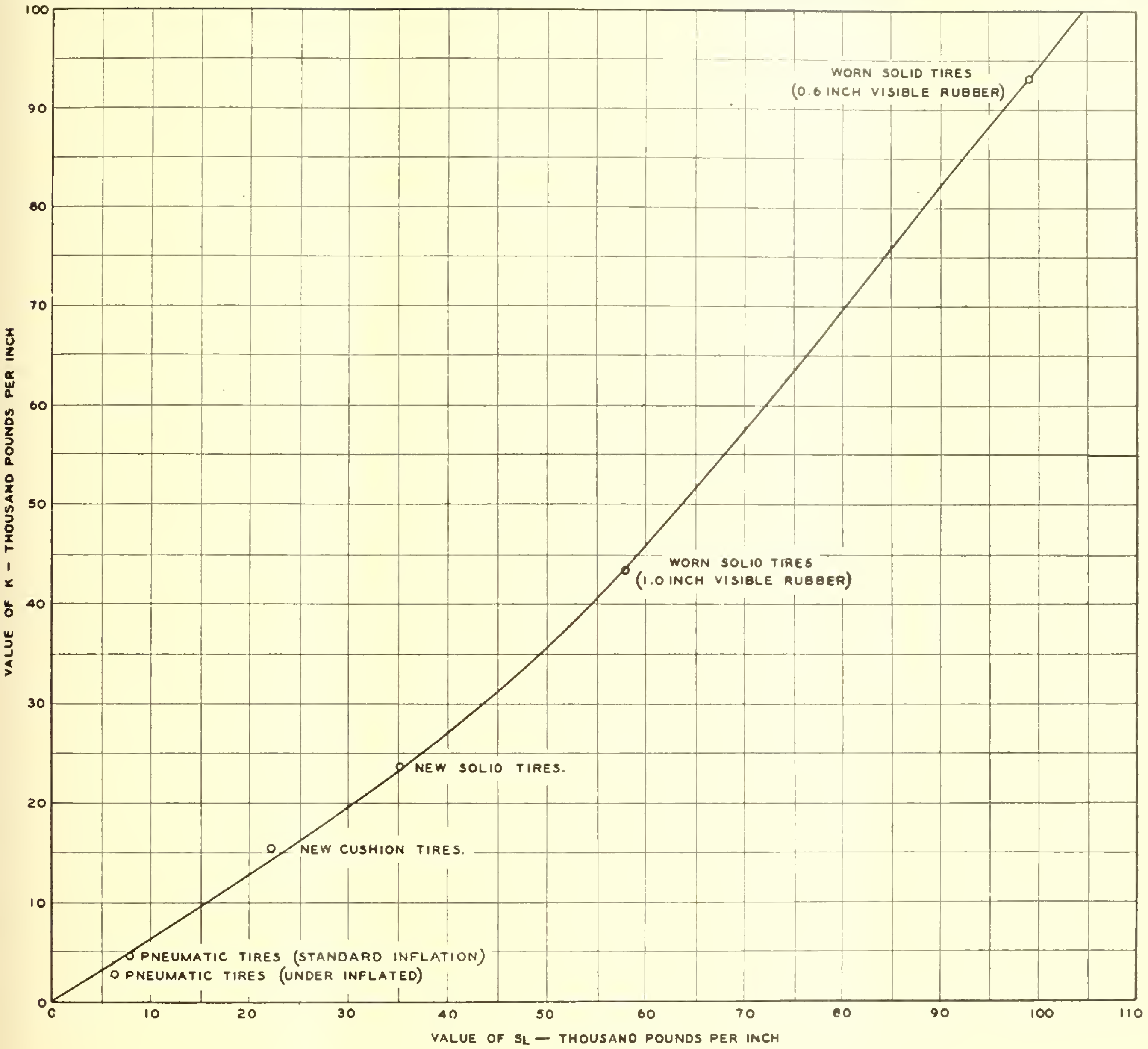

Figure 31.—Relation Between K and S_L

In Figure 31 the values of K so obtained are plotted against S_L, the loading rate of the tire at capacity load. It is plain that a relation exists between these two quantities. The curve drawn in this figure is taken as a basis for estimating shock periods when the test conditions are known.

Estimation of the period of impact by the methods described above makes possible the use of the n vs. c' relation of Figure 23 in computing impact accelerations from the records of the coil spring accelerometer. It is desirable also to take account of the probable dispersion of results obtained from this instrument as shown in Figure 26. For this purpose the curves e_{50} and e_{90} are used. From the n vs. c' relation is obtained the most probable value of acceleration, the probable error of which is given by the ordinate on the curve e_{50} corresponding to the value of R obtained in the given test. The curve e_{90} indicates the approximate extreme range of error to be expected.

CONSTANT VALUE OF c CAN BE USED UNDER CERTAIN CONDITIONS

The fact has been noted that for values of n between 1.0 and 3.5 the value of c varies but little. This fact has led to the adoption of a constant value, $c = 0.65$, for use under conditions where there is reasonable assurance that values of n to be encountered lie within those limits. Effective use of this constant requires a knowledge of the range of periods to be expected and a careful choice of the accelerometer element to be used. The dispersion of the values will be similar to that encountered when the n vs. c relation is used, but somewhat greater, because of the small error in c.

STUDY MADE OF ACCURACY OF THE DATA OBTAINED IN THE PREVIOUSLY CONDUCTED MOTOR TRUCK IMPACT TESTS

One of the primary objectives of this instrument investigation was to determine the accuracy of the motor-truck impact data upon which the published report and the two reports withheld from publication, are based. It has been found that the magnitude of the reaction depends upon four major variables (tire equipment, wheel load, vehicle speed, and road roughness conditions) and a consideration of the accuracy of the data should comprehend suitable ranges of these variables. Such a consideration should also include a fair range in accelerometer elements which were used in gathering the data. The best available material which fulfills these requirements is found in the tests which formed the basis for Figures 12 to 16 in the report entitled "Motor Truck Impact as Affected by Tires, other Truck Factors, and Road Roughness," published in the June, 1926, issue of PUBLIC ROADS. A comparison of the original curves with curves plotted from recomputations made in the full light of this accelerometer investigation will, it is believed, be a fair measure of the general accuracy of the motor truck impact data. Table 8 gives the variation in the test conditions for the data selected for recomputation.

TABLE 8.—*Schedule of the test conditions for which the data were recomputed*

Tire	Accelerometer characteristics		Tests over artificial obstructions			Road tests
	$\frac{gs}{w}$	T_2	Speed variation	Load variation	Obstructions [1]	Road, speed, and load [2]
	Feet per second per second per inch	*Seconds*	*Miles per hour*	*Pounds*		
Dual pneumatic (No. 47).	832	0.0629	5–22½	2,365–6,600	1d, 7d, 2d 4s, 8s, 3s 4d, 8d, 3d	R and T, 12 miles per hour, 4,400 pounds.
Dual new cushion (Nos. 13 and 16).	1,748	.0434	5–22½	2,445–5,100	1d, 7d, 2d 4s, 8s, 3s 4d, 8d, 3d	R and T, 12 miles per hour, 3,400 pounds.
Dual new solid (Nos. 40 and 41).	2,382 3,594 4,892	.0372 .0303 .0259	5–20	2,349–6,000	1d, 7d, 2d 4s, 8s, 3s 4d, 8d, 3d	R and T, 12 miles per hour, 4,000 pounds.
Dual worn solid (Nos. 40a and 41a).	2,382 4,892 7,824	.0372 .0259 .0205	5–17½	2,317–6,000	1d, 7d, 2d 4s, 8s, 3s 4d, 8d, 3d	R and T, 12 miles per hour, 4,000 pounds.

[1] 1d, 7d, 2d: Drop from 30-inch inclined planes, 0.81, 1.50, 1.94 inches high, respectively. 4s, 8s, 3s; and 4d, 8d, 3d: Respectively shock at and drop after 3-inch rectangular obstructions, 0.56, 0.88, 1.12 inches high, respectively.

[2] R and T: Rough stone and smooth concrete road sections, respectively.

The period T_1 for each of the test conditions involving artificial obstructions was computed by the method outlined in preceding paragraphs. It has been found that, within the operating conditions of the impact tests, the drop impact is more severe than the shock. Since the road tests were analyzed so as to show the magnitude with respect to the frequency of occurrence the greater significance was given to the more severe impacts. For the reason given above these were assumed to be drop impacts and the period T_1 used in recomputing the data for road tests was therefore computed on the drop basis.

DATA RECOMPUTED, WITH DISPERSION ZONES INDICATED

The values of T_1 having been estimated, the values of the ratio $n=\frac{T_1}{T_2}$ were computed for each test condition. For each value of n, the calibration coefficient $c=\frac{C}{\frac{gs}{w}}$ was obtained from the curve given in Figure 23 and the calibration factor C for each test condition was computed. Using these individual calibration factors the recomputations were made by substituting the new values for C in the equation for total impact reaction

$$F=mCR+mg+S, \qquad (13)$$

wherein m = unsprung mass,

R = accelerometer reading,[5]
g = acceleration due to gravity,
S = truck spring component.

The impact reaction is expressed as a percentage of the static wheel load by dividing by the corresponding static loads and multiplying by 100.

---

[5] The field data were originally plotted on work sheets so that the accelerometer readings could be interpolated at the speeds selected for computation. The interval in speed between the plotted points was about 3 miles per hour. It is these interpolated values of accelerometer reading which were used in the original computations and in these recomputations.

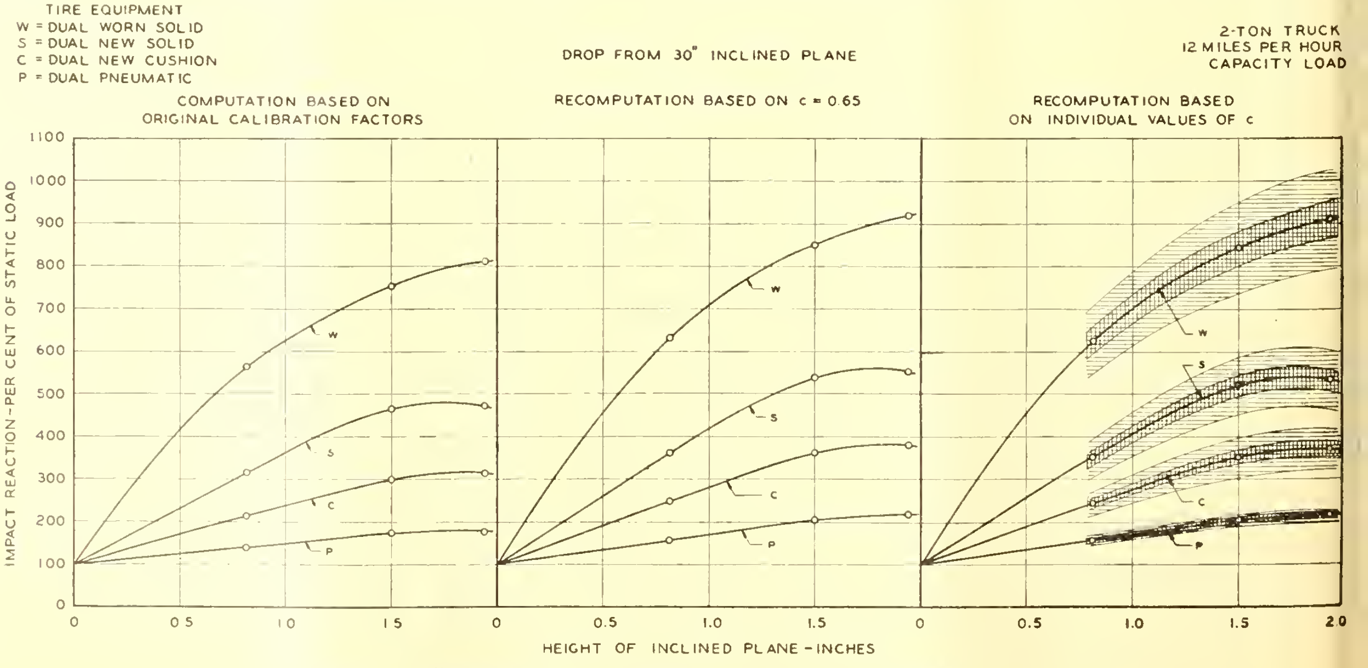

FIGURE 32.—COMPARISON OF ORIGINAL AND RECOMPUTED IMPACT DATA

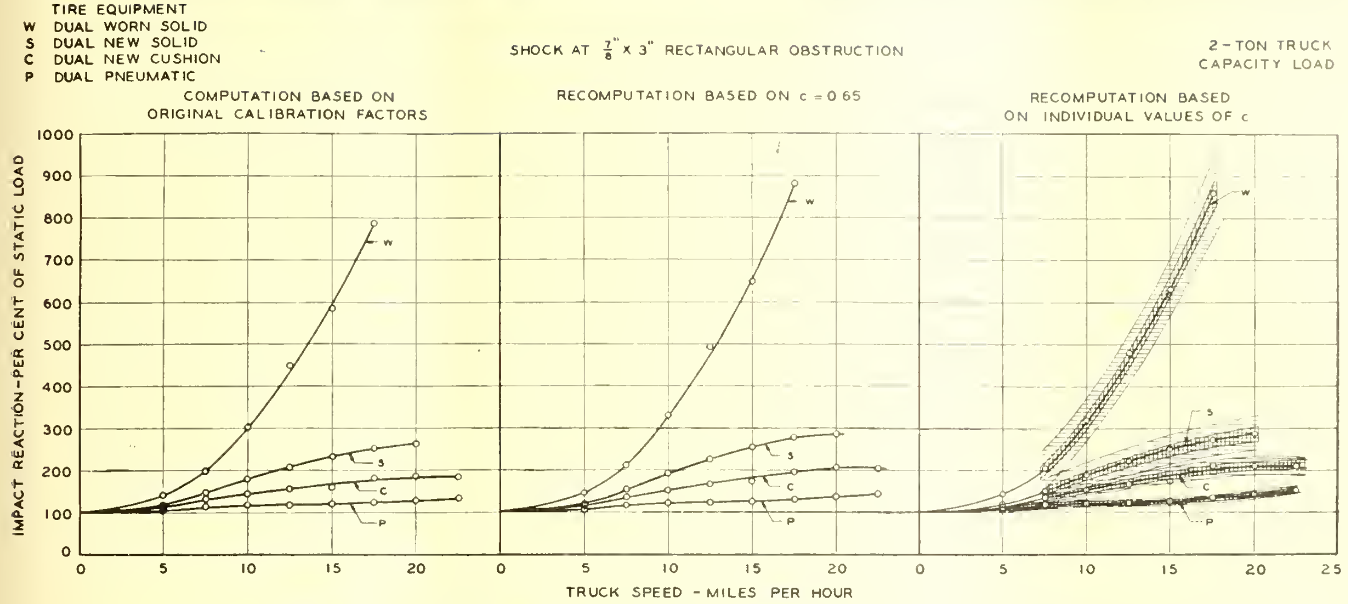

Figure 33.—Comparison of Original and Recomputed Impact Data

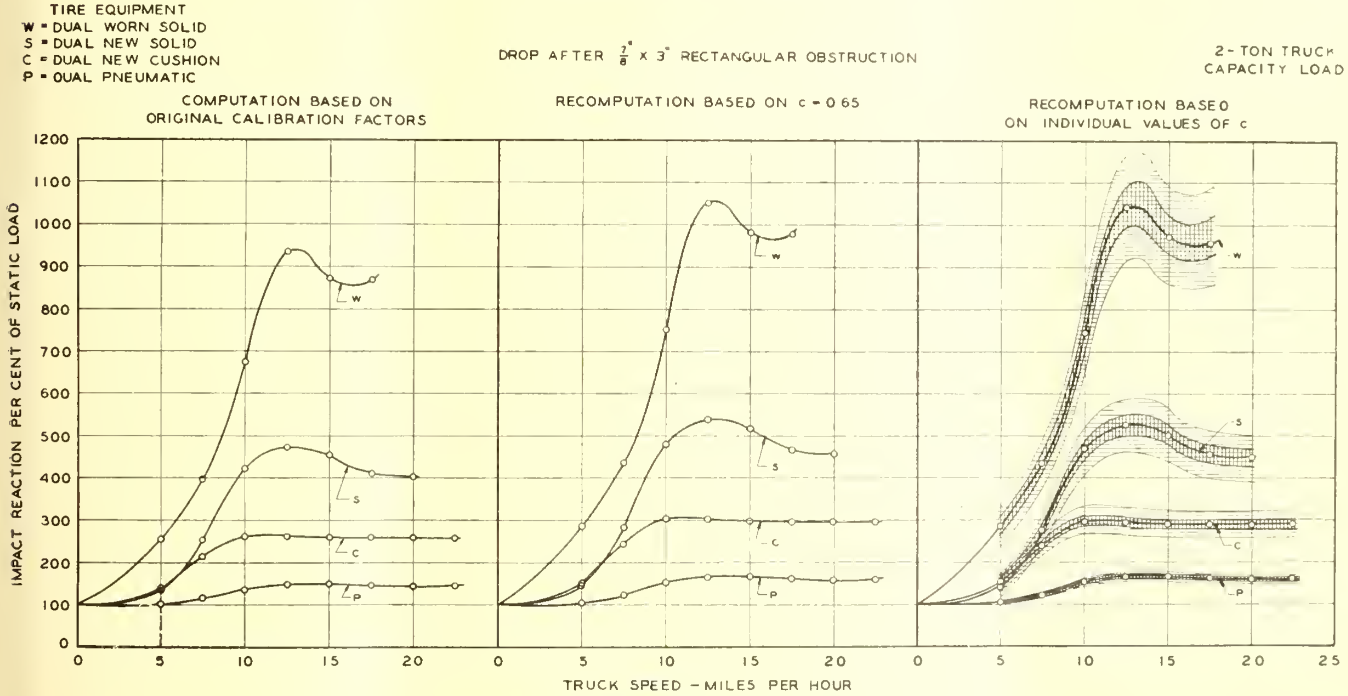

Figure 34.—Comparison of Original and Recomputed Impact Data

In a preceding paragraph it was shown that there is an instrumental error due to dispersion and that the error bears a distinct relation (fig. 26) to the magnitude of the accelerometer reading. The indicated dispersion zones applicable to the recomputed impact data were computed for each test condition using actual values of the reading, R, to obtain values of the error e_{50} and e_{90}. It will be noted that these errors affect only the term mCR in equation 13.

The study of the n vs. c relation also showed that a constant value of c equal to 0.65 would reasonably apply throughout the range in n ordinarily encountered in motor-truck impact tests. The use of such a constant calibration factor greatly simplifies the computation. The values of impact reaction were also recomputed on this basis ($c = 0.65$) and the results are shown in conjunction with the results by the original computation method and the results of the computation method involving an individual calibration factor for each test condition, in order to compare the impact relations obtained when the same field data are computed according to the three methods.

Figures 32 to 35 have been selected to show the order of variation caused by the three computations. In each figure the left-hand panel represents the data as originally computed. The center panels represent recomputed values obtained with calibration factors based on a constant value of c equal to 0.65. The right-hand panels represent the recomputed values obtained with individual values of c according to the n vs. c relation shown in Figure 23. The 50 per cent and 90 per cent dispersion zones are shown in vertical

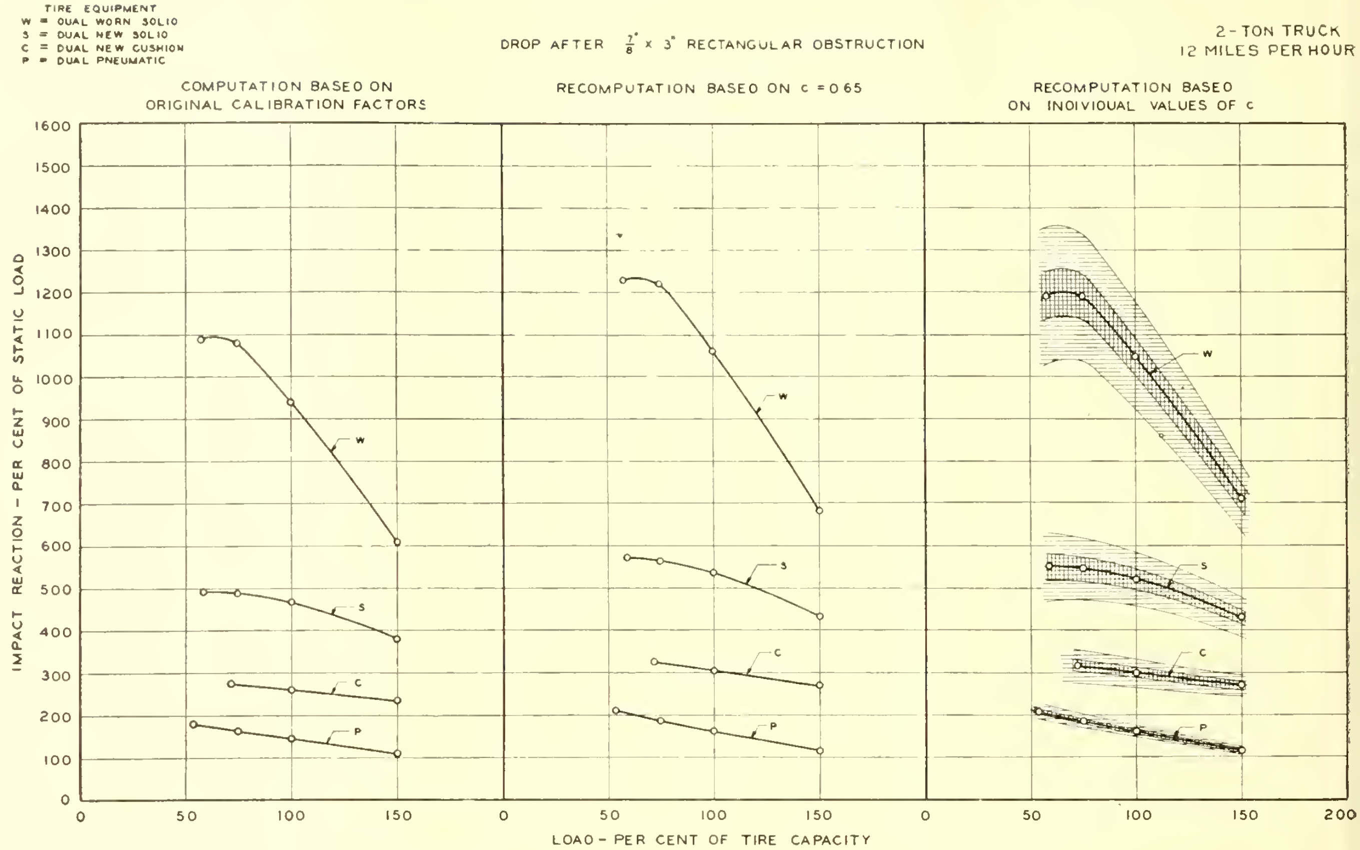

Figure 35.—Comparison of Original and Recomputed Impact Data

and horizontal shading, respectively, superimposed on the curves in the last named panels. It is quite evident that general conclusions as to the influence of various factors on motor truck impact reactions could be determined equally well by the curves according to any of the three methods of computation.

There is a tendency, however, for the recomputed impact reactions to be higher than the originally computed values. This is better shown in Figures 36 and 37, in which the original data have been plotted against corresponding recomputed values according to the two methods of recomputation ($c=0.65$ and c having individual values, respectively). In these figures all the data available have been plotted, 56 shock conditions and 112 drop conditions being represented. Figure 36 shows that the recomputed values obtained on the basis of $c=0.65$ are about 12½ per cent higher than the original values while the recomputed values obtained by the use of individual calibration factors (see fig. 37) averaged about 11½ per cent higher than the original.

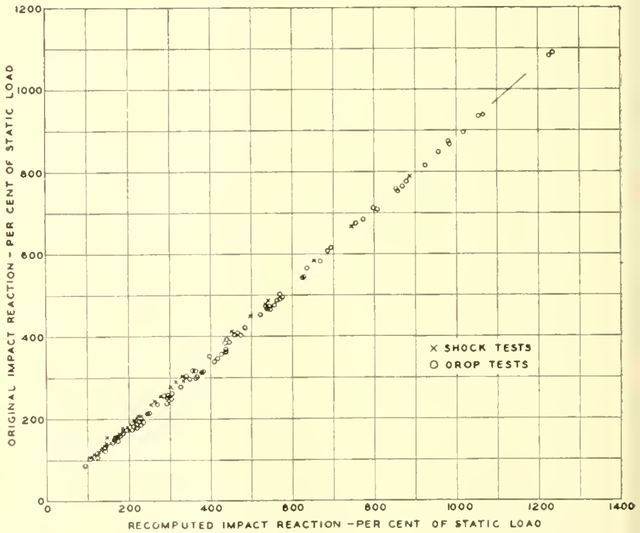

Figure 36.—Comparison of Original and Recomputed Impact Data. Recomputed Values on the Basis of $c=0.65$

An inspection of the right-hand panels in Figures 32 to 35 shows that the dispersion apparently varies directly with the magnitude of the impact reaction. This is better illustrated in Figure 38 which was constructed from work sheets similar to Figure 37 but on which the recomputed values were represented by lines defining the respective dispersion bands. The areas enveloping these bands have been shown in Figure 38, the vertical shading representing the 50 per cent dispersion zones and the horizontal shading representing the 90 per cent dispersion zones for the shock and drop conditions. The magnitudes of the dispersion ranges indicated in Figure 38 are expressed below as percentages of the most probable recomputed value i. e., the value of the center of the dispersion zones:

	Shock, per cent	Drop, per cen
50 per cent range	6	
90 per cent range	13	1

In the discussion of Figure 37 it was shown tha the recomputed or most probable values of impac reaction differ from the original values by an averag percentage equal to 11.5. We may, therefore, conclud

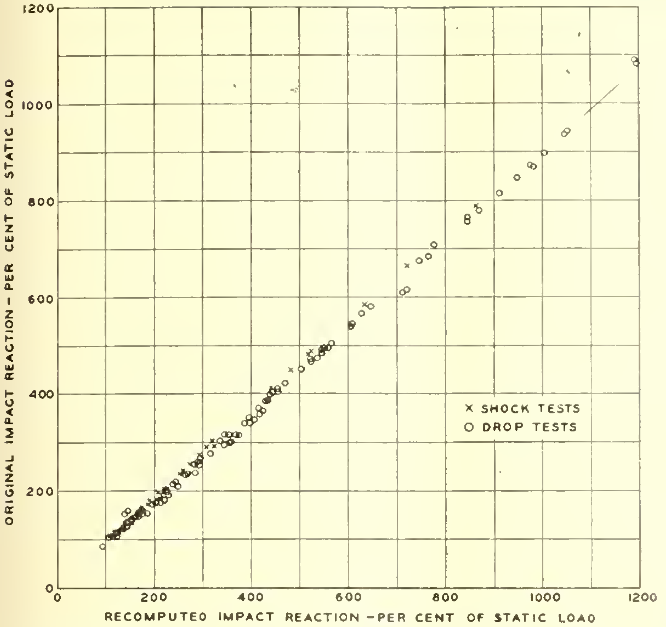

Figure 37.—Comparison of Original and Recomputed Impact Data. Recomputed Values Based on Individual Calibration Factors

the discussion of the reaction wheel tests with the observation that impact forces based upon the previous calibration are systematically in error by about 12 per cent. The coil spring accelerometer is subject to dispersion errors which decrease in magnitude with an increase in deflection. For the significant accelerations measured with a given spring-weight combination, 50 per cent of the values of measured acceleration lie within 8 per cent of the mean and 90 per cent within 15 per cent of the mean. These values were obtained by tests on the reaction wheel involving artificial obstructions and the committee interprets them as applying also to motor-truck impact tests on a smooth road with artificial obstructions. For motor-truck tests on naturally rough roads greater uncertainties exist.

CONCLUSIONS OF ACCELEROMETER COMMITTEE SUMMARIZED

In summarizing its detailed report the committee called attention to the fact that its conclusions are based on a careful analysis of results obtained within limited ranges of speed and other operating conditions and that these conclusions should not be assumed to apply beyond these ranges. The conclusions reached by the committee are as follows:

1. The contact type of accelerometer may be used to obtain highly accurate measurements of acceleration of the order of magnitude and period encountered in motor-truck impact.

(*a*) Single-element instruments of this type are suitable for use under conditions where a given phenomenon is periodically reproduced such as by a drop machine or a reaction wheel. The cantilever-spring contact accelerometer was investigated and is considered to be satisfactory by this committee.

(*b*) While this committee has not investigated a multiple-element cantilever-spring contact accelerometer, it is of the opinion that such an instrument may be developed for use in road tests.

2. The coil spring accelerometer may be used to obtain reasonably accurate measurements of acceleration due to motor truck impact when due consideration is given to the relation between the period of the impact reaction and the period of the accelerometer element.

(*a*) With these restrictions instruments of this type are suitable for road tests. Their operation is not difficult and results are quickly obtained with them.

(Continued on page 111)

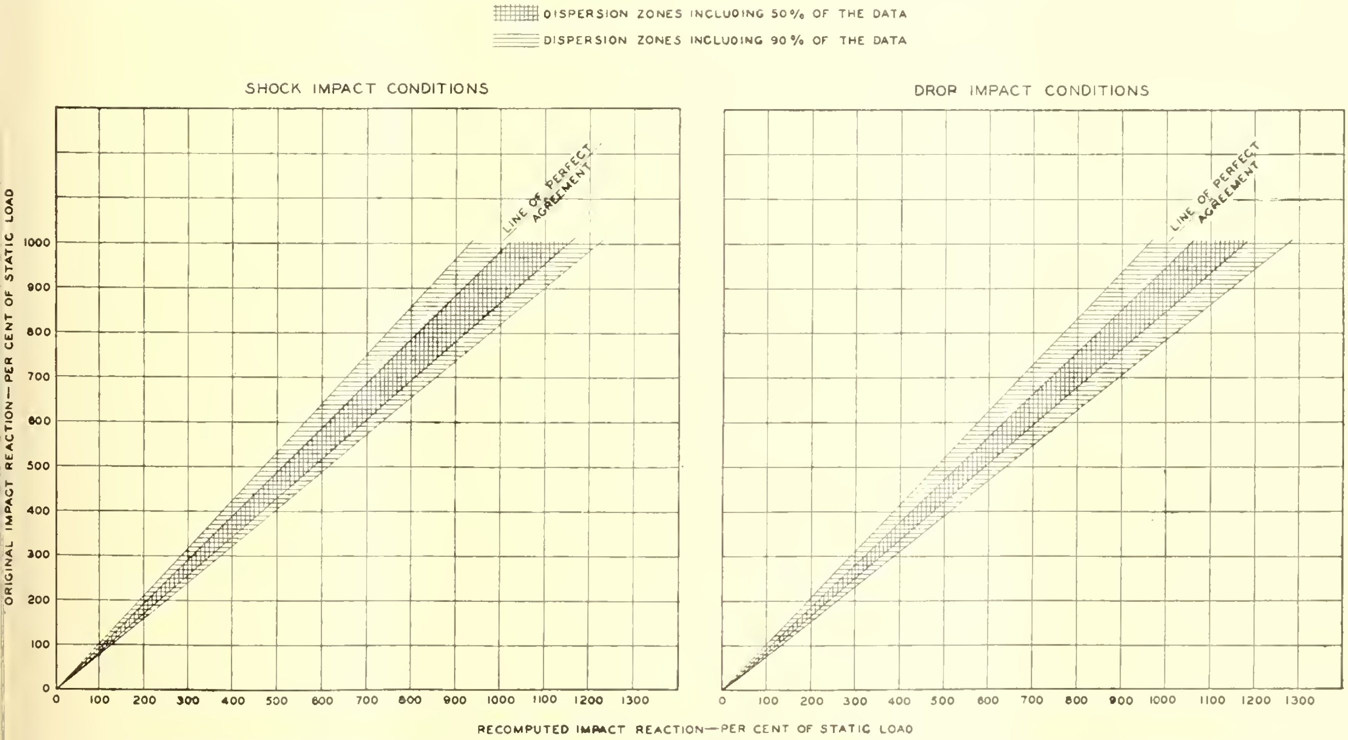

Figure 38.—Agreement Between Originally Computed Impact Reactions of the Motor Truck Tests and Corresponding Recomputed Reactions

PROGRAM FOR INTERNATIONAL ROAD CONGRESS ANNOUNCED

Official delegates from 50 countries in all parts of the world have announced that they will attend the Sixth Congress of the Permanent International Association of Road Congresses which will open in Washington on Monday, October 6, and close on Saturday, October 11. Large delegations are expected from England, France, Germany, and Italy, and it is possible that some of these delegations may include as many as 100 engineers. Smaller delegations from other countries, including those as far distant as China, India, and Australia, will make the total attendance large.

This is the first such congress ever to be held outside of Europe, and it is believed that it will be the largest and most important gathering of highway engineers ever held. Contributing to this end, there is now a world-wide interest in highway transportation and a desire by foreign engineers to inspect the methods of construction and the results which have been attained in the United States. The delegations will include the most influental and representative of foreign highway engineers.

In preparation for the congress, 69 papers from engineers in 20 different countries are being published and will be received by the members of the congress before they start for Washington. These papers deal with the following subjects, which have been included in the agenda of the congress:

FIRST SECTION

CONSTRUCTION AND MAINTENANCE

First question: Results obtained by the use of—
- (*a*) Cement.
- (*b*) Bricks or other artificial paving.
  (Methods employed for road construction and maintenance in these materials.)

Second question: The most recent methods adopted for the use of tar, bitumen, and asphalt in road construction.

Third question: The construction of roads in new countries, such as colonies and undeveloped regions.

SECOND SECTION

TRAFFIC AND ADMINISTRATION

Fourth question: Ways and means of financing highways:
- (*a*) Road construction.
- (*b*) Maintenance.

Fifth question: Highway transport: Correlation and coordination with other methods of transport; adaptation to collective (organizations) and individual uses.

Sixth question:
1. Traffic regulation in large cities and their suburbs; traffic signals; design and layout of roads and adaptation to traffic requirements in built-up areas.
2. Parking and garaging of vehicles.

The reports on each question have been reviewed by a general reporter (two general reporters for first question), and these general reports will be read at the meeting of the congress, followed by open discussion and adoption of conclusions. It is the custom to appoint general reporters from the country in which the congress is held, and the following authorities of recognized standing are acting in this capacity: Question 1, part (*a*), Frank T. Sheets, chief highway engineer, Illinois Department of Public Works; question 1, part (*b*), P. J. Freeman, chief engineer, bureau of tests and specifications, department of public works, Allegheny County, Pittsburgh, Pa.; question 2, Roy W. Crum, director, highway research board, National Research Council, Washington, D. C.; question 3, Edwin W. James, chief, Division of Highway Transport, Bureau of Public Roads, United States Department of Agriculture, Washington, D. C.; question 4, A. B. Barber, manager, transportation and communications department, Chamber of Commerce of the United States of America, Washington, D. C.; question 5, Henry R. Trumbower, professor of economics, University of Wisconsin, Madison, Wis., and question 6, Miller McClintock, director, Albert Russel Erskine Bureau for Street Traffic Research, Harvard University, Cambridge, Mass.

PROGRAM OF THE CONGRESS

The American Organizing Commission has announced the following program of the congress:

Monday, October 6

Morning: Meeting of Permanent International Commission. Registration of delegates.

Afternoon: First plenary session—opening of congress—Constitution Hall.

Evening: No formal engagement.

Tuesday, October 7

Morning: Section meetings.

Luncheon: American Road Builders' Association.

Afternoon: Official opening of International Exposition of American Road Builders' Association.

Evening: Visits to International Exposition.

Wednesday, October 8

Morning: Section meetings.

Afternoon: Section meetings.

Evening: Official reception.

Thursday, October 9

Morning: Section meetings.

Afternoon: Second plenary session for discussion of conclusions.

Evening: American Organizing Commission dinner.

Friday, October 10

Morning: Inspection trip to experiment station of the United States Bureau of Public Roads at Arlington, Va.

Luncheon: Picnic luncheon, auspices of American Organizing Commission, at Mount Vernon, Va.

Afternoon: Inspection of construction of Mount Vernon Memorial Highway and visit to Mount Vernon (the home of George Washington).

Evening: Official closing of the congress and reception.

Saturday, October 11

Morning: Inspection of United States Naval Academy at Annapolis, Md., under auspices of American Organizing Commission.

Luncheon: Reception and luncheon tendered by the Hon. Albert C. Ritchie, Governor of Maryland.

Afternoon: Recreation.

Evening: No formal engagement.

NOTE.—The necessity may arise for certain modfications in the program as here given.

Participation in all official excursions and social unctions will be by ticket, which will be issued without harge to all delegates from other countries and to fficial delegates from the United States.

The congress is being held in this country at the nvitation of the United States Government and rrangements are being made by the American Organizng Commission with headquarters at 1723 N Street NW., Washington. The commission consists of: President, Roy D. Chapin, National Automobile Chamber f Commerce; Secretary-General, Thomas H. MacDonald, United States Bureau of Public Roads; and he following members, Wilbur J. Carr, Department f State; A. J. Brosseau, Chamber of Commerce of he United States; H. H. Rice, Highway Education Board; Robert Hooper, American Automobile Assocition; H. G. Shirley, American Association of State Highway Officials; Thomas R. Taylor, Department of Commerce; and Charles M. Upham, American Road Builders' Association. The commission is assisted by Pyke Johnson, National Automobile Chamber of Commerce, and H. S. Fairbank, United States Bureau of Public Roads, as administrative aides, and J. Truman Thompson, highway research specialist, United States Bureau of Public Roads, as manager.

Participation in the proceedings of the congress and eceipt of the reports to the congress and the printed roceedings of the congress are possible only through membership in the congress. A temporary memberhip carrying these privileges may be had upon applicaion to the American Organizing Commission at the bove address. The fee for such membership is $5.

Simultaneously with the congress, an international oad machinery and materials exhibition and demontration will be held by the American Road Builders Association. The exhibition will be held in the Washngton Auditorium, while the demonstration grounds where machinery may be seen in operation will be rovided at a near-by point.

AMERICAN ENGINEERS URGED TO ATTEND CONGRESS

It is hoped that a large number of American highway engineers and representatives of related industries will attend the congress and participate in the proceedings. The attractive program of meetings and trips which has been arranged and the opportunity of meeting the leaders in highway construction and transportation should attract a large attendance. October is one of the most desirable months of the year for a visit to Washington.

(Continued from p. 109)

(*b*) The coil spring accelerometer is subject to dispersion errors the magnitude of which decreases as the length of record increases. However, the indicated probable errors in total force resulting from this dispersion are not excessive for the type of tests involved.

3. A recomputation of published data using calibration factors obtained in this investigation indicates that the impact reaction values which are based on the original calibration factors are from 10 to 15 per cent too low because of systematic errors in calibration.

The comparisons made and conclusions drawn in reports based on such data, being general in nature and depending upon many tests rather than upon individual measurements, show with sufficient accuracy the relations and factors which influence the magnitude of the impact reactions.

4. The displacement-time apparatus may be used to determine the magnitude of the acceleration, the period of impact, and other characteristics of motor-truck impact reactions.

(*a*) Its use is limited to laboratory set-ups.

(*b*) Good agreement has been secured between acceleration values determined by the analysis of displacement-time curves, those measured with an accelerometer of the contact type, and those computed from a simple harmonic motion machine.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

JUNE 30,1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,153.8	$ 2,326,218.40	$ 1,148,172.11	83.2	22.2	105.4	$ 13,545.40	$ 6,772.70				$ 4,083,610.18	Alabama
Arizona	810.3	4,114,720.89	3,088,393.26	136.5	149.1	285.6	116,572.12	87,787.12		15.8	15.8	1,898,927.83	Arizona
Arkansas	1,741.4	5,523,331.76	2,530,094.42	142.9	46.2	189.1	1,759,692.74	879,846.76	86.4	3.5	89.9	1,518,012.42	Arkansas
California	1,880.2	7,170,064.91	2,901,983.78	119.4	27.8	147.2	1,577,783.43	698,070.27	36.1	2.1	38.2	1,743,512.14	California
Colorado	1,208.1	4,989,906.97	2,314,893.56	193.6	28.3	221.9	1,232,562.53	617,597.98	29.0	69.0	98.0	2,283,653.54	Colorado
Connecticut	243.3	2,351,390.33	1,047,625.20	10.9		10.9	446,379.45	117,900.00	7.9		7.9	714,247.71	Connecticut
Delaware	261.0	900,925.60	357,666.32	24.9		24.9	732,886.27	362,874.26	41.2		41.2	91,061.91	Delaware
Florida	503.5	5,127,463.61	2,358,944.67	97.1	5.5	102.6						1,681,203.79	Florida
Georgia	2,703.3	3,140,227.85	1,525,964.76	118.6	32.2	150.8	2,962,085.96	1,360,367.13	59.6	68.4	128.0	2,902,272.18	Georgia
Idaho	1,194.1	1,315,433.28	793,224.26	85.2	27.8	113.0	996,404.66	541,676.25	54.3	42.4	96.7	1,175,024.06	Idaho
Illinois	2,056.1	15,980,580.76	7,088,161.17	454.4		454.4	6,282,202.65	2,808,075.49	163.9	63.5	217.4	4,666,255.25	Illinois
Indiana	1,481.6	4,953,074.92	2,363,937.39	163.2		153.2	241,360.20	120,680.10	9.9		9.9	2,473,551.68	Indiana
Iowa	2,979.7	7,129,677.69	3,036,286.94	66.0	176.9	242.9	1,140,139.79	475,042.91	11.8	27.8	39.6	1,892.77	Iowa
Kansas	2,833.9	5,640,308.69	2,707,334.03	248.7	27.8	276.5	645,979.77	321,602.02	20.6	89.5	110.1	2,063,112.86	Kansas
Kentucky	1,530.2	3,736,246.74	1,638,751.70	121.6	5.5	127.1	4,738,366.79	2,265,599.85	50.5	228.7	279.2	114,878.62	Kentucky
Louisiana	1,352.6	4,992,472.59	2,432,542.49	153.8	14.3	168.1	1,654,549.25	805,908.66	49.0	11.4	60.4	1,224,288.48	Louisiana
Maine	534.8	2,327,133.99	869,119.44	57.6		57.6	934,183.56	326,809.63	23.9	1.5	25.4	1,354,501.03	Maine
Maryland	630.7	1,484,021.60	706,596.93	34.8	12.6	47.4	1,213,068.76	571,838.21	26.0	5.8	31.8	5,543.03	Maryland
Massachusetts	667.4	4,410,279.29	1,389,811.54	66.0	2.6	68.6	2,465,209.61	444,715.17	11.3		11.3	1,944,255.39	Massachusetts
Michigan	1,602.0	10,006,110.54	4,233,567.45	230.6	30.4	261.0	824,037.67	363,276.00	30.1		30.1	2,782,145.14	Michigan
Minnesota	3,936.1	10,791,550.43	3,819,278.31	232.2	237.5	469.7	1,232,803.68	490,675.40	19.4	32.3	51.7	46,352.03	Minnesota
Mississippi	1,820.7	1,795,493.20	691,571.41	55.0	7.7	62.7	48,835.05	24,417.52	.1		.1	3,527,965.68	Mississippi
Missouri	2,486.8	7,687,851.27	2,665,580.20	117.6	61.3	178.8	3,642,322.65	1,248,339.72	61.6	27.7	89.3	831,704.84	Missouri
Montana	1,717.4	7,874,376.59	4,608,614.27	526.9	43.7	569.6	1,120,677.78	641,330.56	90.2	40.7	130.9	3,540,466.15	Montana
Nebraska	3,669.2	7,382,359.82	3,442,561.75	272.9	146.9	418.8	1,578,049.64	636,872.07	54.7	91.1	146.8	2,176,337.14	Nebraska
Nevada	1,219.2	802,672.12	713,174.12		122.1	122.1	348,831.87	307,040.69		81.0	81.0	773,969.02	Nevada
New Hampshire	352.7	1,542,914.69	552,098.03	36.6		36.6						221,369.45	New Hampshire
New Jersey	507.9	5,923,349.51	1,476,271.32	66.7		66.7						977,245.76	New Jersey
New Mexico	1,904.4	3,711,585.77	2,433,941.53	185.3	50.7	236.0	643,139.21	479,303.62	32.6	1.9	34.6	969,531.76	New Mexico
New York	2,491.0	22,301,867.76	4,499,355.00	300.4		300.4	7,348,800.00	1,261,672.50	83.6		83.6	7,315,347.86	New York
North Carolina	1,780.5	3,352,750.01	1,652,180.40	146.6	28.9	175.5	845,693.09	401,143.99	25.8	1.9	27.7	2,660,469.31	North Carolina
North Dakota	4,262.5	1,987,394.70	1,057,103.34	340.6	127.8	468.3	1,221,468.57	605,444.83	161.3	207.1	368.4	1,452,524.56	North Dakota
Ohio	2,186.7	20,373,300.95	6,579,243.04	367.8	35.2	403.0	6,055,233.63	1,732,197.73	83.1	13.1	96.2	939,205.63	Ohio
Oklahoma	1,890.4	4,057,669.37	1,807,476.30	116.4	56.2	172.6	3,192,412.33	1,482,314.49	92.7	49.7	142.4	184,302.92	Oklahoma
Oregon	1,150.4	5,343,938.89	3,150,289.24	204.6	85.4	289.9	819,262.95	490,778.04	49.1		49.1	288,573.31	Oregon
Pennsylvania	2,341.9	16,972,451.08	4,590,035.23	211.7	14.1	225.8	6,603,495.13	1,978,331.84	87.1		87.1	1,082,376.94	Pennsylvania
Rhode Island	184.8	1,960,716.15	668,452.68	28.2		28.2						588,570.42	Rhode Island
South Carolina	1,868.5	4,495,500.70	1,927,574.94	98.3	80.3	178.6	1,463,356.75	629,064.20	27.2	26.2	53.4	97,283.99	South Carolina
South Dakota	3,445.1	4,526,179.47	2,422,856.94	454.1	142.0	596.1	586,201.87	368,755.17	44.0	70.6	114.6	1,062,892.43	South Dakota
Tennessee	1,260.9	3,278,768.07	1,518,961.84	132.3	12.5	144.8	2,817,321.13	1,107,295.27	82.3	38.2	120.5	1,552,381.84	Tennessee
Texas	6,835.6	12,367,394.51	5,019,494.74	376.3	119.8	496.1	2,946,441.96	1,207,057.26	101.6	52.7	154.3	4,647,819.57	Texas
Utah	981.1	1,172,923.88	810,322.47	56.3	10.8	67.1	704,739.73	516,652.56	44.9	78.8	123.7	662,833.17	Utah
Vermont	265.6	2,225,996.90	773,211.81	42.5	2.6	45.1	316,892.92	45,829.37	5.7		5.7		Vermont
Virginia	1,467.9	4,290,724.92	2,021,632.97	203.1	13.6	216.7	833,708.70	395,539.94	28.0		28.0	691,693.83	Virginia
Washington	901.8	3,992,309.81	1,713,300.00	96.5	29.8	126.3	248,056.42	144,900.00	2.1	6.5	8.6	1,211,773.29	Washington
West Virginia	710.1	3,474,452.68	1,322,067.98	76.0	27.8	102.8	1,184,345.87	386,995.23	20.5	17.2	37.7	628,588.82	West Virginia
Wisconsin	2,246.1	7,729,781.66	3,114,270.98	197.6	48.1	245.7	992,121.83	424,875.00	28.0		28.0	816,349.40	Wisconsin
Wyoming	1,708.7	2,122,570.69	1,382,750.59	148.5	92.6	241.1	453,127.23	339,821.63	13.7	63.2	76.9	588,414.28	Wyoming
Hawaii	41.2	853,565.90	359,459.43	21.5		21.5	226,820.16	113,414.57	9.2		9.2	1,568,624.59	Hawaii
TOTALS	93,975.1	272,012,001.81	111,630,191.38	7,709.2	2,205.6	9,914.8	76,450,170.75	30,526,282.50	1,940.0	1,529.3	3,469.3	75,716,790.80	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 6 ▽ AUGUST, 1930

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 11, NO. 6 AUGUST, 1930 G. P. St. CLAIR, Editor

TABLE OF CONTENTS

Page

Freezing and Thawing of Soils as Factors in the Destruction of Road Pavements . . . 113

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FREEZING AND THAWING OF SOILS AS FACTORS IN THE DESTRUCTION OF ROAD PAVEMENTS

By STEPHEN TABER, Professor of Geology, University of South Carolina, Columbia, S. C.

Soils, when frozen under natural conditions, generally behave as open systems with respect to water. The extent of heaving may be either greater or less than that encountered when freezing occurs in a closed system. Some soils, under certain conditions, freeze with no appreciable uplift of the surface, while others give uplifts as great as 60 per cent of the depth of freezing. Soils that are very impermeable because of high colloid content behave in laboratory tests essentially as closed systems. These include certain muck and gumbo soils, and soils containing bentonite.

Laboratory experiments show that excessive heaving is always accompanied by the segregation of ice in layers or lenses. Similar segregations have been observed when excavations were made in badly heaved ground. The factors which chiefly affect ice segregation are texture of soil, composition of soil, supply of water, rate of removal of heat, and surface load.

Various measures are suggested for the elimination or reduction of damage due to ground freezing, thawing, and alternate freezing and thawing. Proper drainage is always essential. Placing a thick layer of coarse material under the pavement, extending to the extreme depth of ground freezing, is an effective but expensive method. Addition of sand to the subgrade will prevent ice segregation. Uniformity of texture in subgrade soils is essential to the avoidance of differential heaving.

Extensive research in the problems of ground freezing is recommended.

FREEZING AND THAWING have caused much damage to road pavements in cold climates, but the processes involved have not been clearly understood, and therefore some of the preventive measures adopted have proved to be of little or no value. It has been generally assumed that the uplift of the surface soil is due to the change in volume which accompanies the freezing of water, and that expansion is upward because of the less resistance in that direction. But these assumptions are founded on experiments in which water was frozen in closed systems; whereas soils when frozen under natural conditions usually behave as open systems with respect to water, and experiments with open systems lead to quite different conclusions. The freezing experiments of early investigators seem to have been limited to closed systems.

In an investigation of frost heaving begun in 1914 the author experimented with open systems. These experiments, which were carried out on cold nights, indicated that the pressure effects accompanying freezing are due to the growth of ice crystals and that excessive heaving is to be explained by the segregation of water as it freezes. This segregation was obtained in clay but not in clean sand. After the publication of brief descriptions of some of the experiments, together with the conclusions mentioned above,[1] the investigation was dropped temporarily, because of lack of facilities for obtaining and maintaining low temperatures.

The investigation was resumed in March, 1927, with an electric refrigerator, which made close control of temperature possible. Since November, 1927, the work has been carried on through the financial cooperation of the United States Bureau of Public Roads. Two papers have resulted from this research; the first[2] being primarily a description of experiments performed in order to determine the effect of different factors on the freezing of soils, while the second[3] was devoted to a discussion of the theory of frost heaving. The present paper contains a description of the apparatus and technique used in the investigation, a brief review of the more important experiments, and a discussion of the ways in which freezing and thawing of soils may affect pavements.

APPARATUS DESCRIBED

The apparatus (see fig. 1) is set up in an unheated basement room, on a cement floor several feet below ground level, so that the room temperature changes very slowly and is practically constant during the experiments, which seldom continue for periods of more than one week. The refrigerator, well insulated with cork packing, is cooled by expansion coils placed across the top of the cabinet. Sulphur dioxide is used in the cooling system, and the compressor, operated by a one-fourth horsepower motor, is placed on a separate foundation so as to eliminate all vibration. The temperature control is automatic. The temperature of the air in the cabinet is recorded by a thermograph and checked by a maximum and minimum thermometer. When tests are made under uniform conditions the temperature can be kept practically constant, except for fluctuations amounting to between 2° and 3° C., which are due to the periodic starting and stopping of the motor.

A box containing sand, placed in the bottom of the cabinet, is insulated on all sides with a packing of glass wool. The soils to be frozen are packed in containers, which are then buried to their tops in the sand, so that freezing takes place from the top down, just as in the ground.

The cold junction of a thermocouple can be buried in the soil at any depth at which a record of the temperature is desired; the warm junction being kept in water containing a Beckman thermometer graduated to hundredths of a degree centigrade. Readings are made on a D'Arsonval reflecting galvanometer. The alternating current operating the motor has no effect on the galvanometer readings; but it is necessary to ground the compressor frame, as otherwise a static charge is set up by the compressor belt which makes accurate readings impossible. The temperature of the soil a short distance below the surface is unaffected by the slight variations in the temperature of the air due to intermittent operation of the compressor, and cooling follows a smooth curve. (See fig. 2.)

In most tests the soils have been frozen in cylindrical pasteboard containers, saturated with paraffin. At the close of the experiment the container is quickly cut away from the frozen soil cylinders, which may then be examined, measured, and photographed. At first gallon-sized containers were used, but later most of the tests were made with standard quart cartons, 16 centimeters in height and 8.5 centimeters in diameter,

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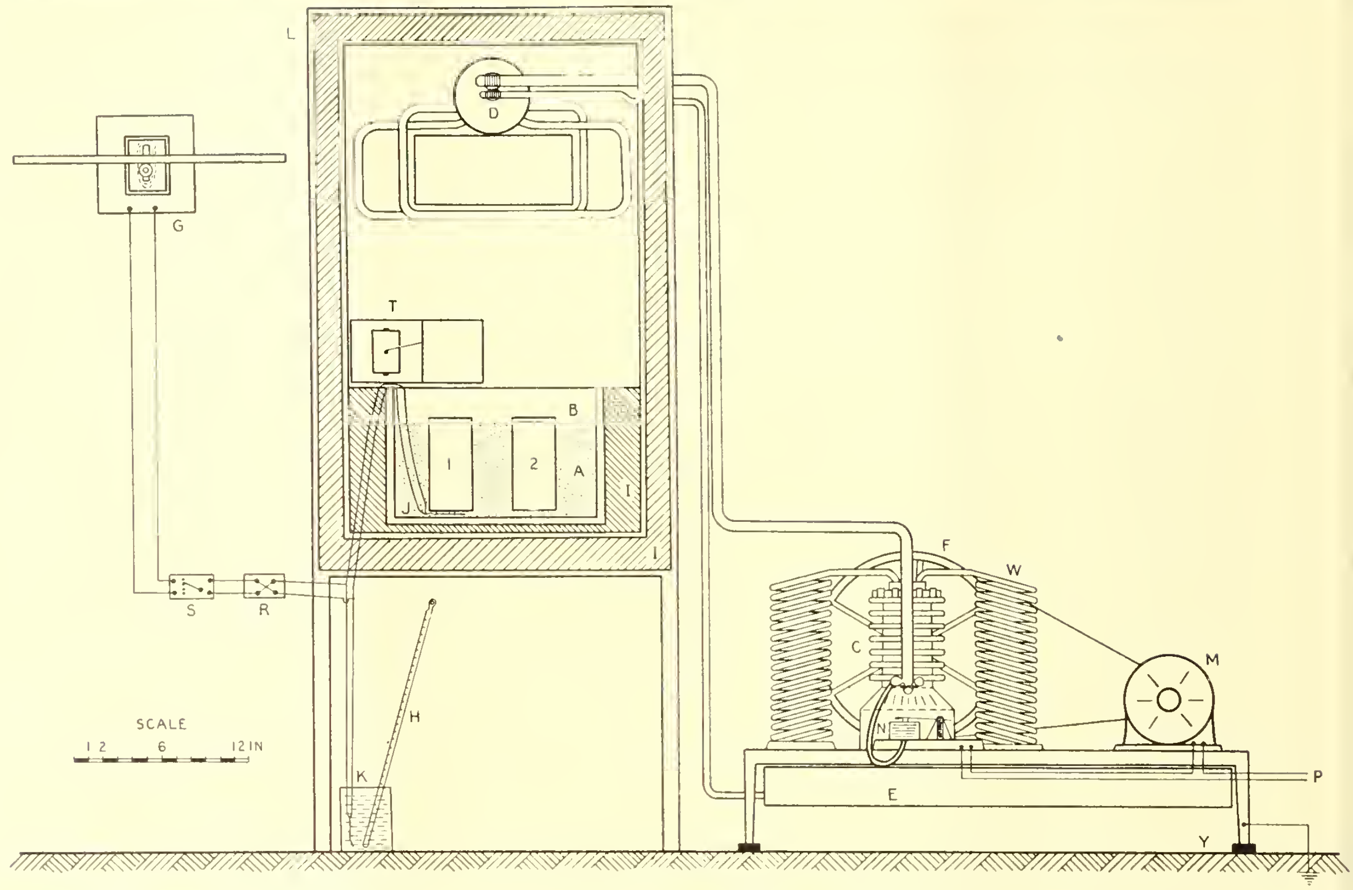
[1] "The Growth of Crystals Under External Pressure," Amer. Jour. of Sci., Vol. XLI (1916), pp. 544-545; "Ice Forming in Clay Soils Will Lift Surface Weights," Eng. News-Record, Vol. LXXX (1918), pp. 262-263; "Surface Heaving Caused by Segregation of Water Forming Ice Crystals," *ibid.*, Vol. LXXXI (1918), pp. 683-684.

[2] "Frost Heaving," Jour. of Geol., Vol. XXXVII (1929), pp. 428-461.

[3] "The Mechanics of Frost Heaving," Jour. of Geol., Vol. XXXVIII (1930), pp. 303-317.



A—Sand
B—Freezing Box
C—Compressor
D—Expansion Drum and Coils
E—SO_2 Reservoir
F—Flywheel and Fan
G—Galvanometer
H—Beckman Thermometer
I—Insulating Packing
J—Cold Junction of Thermocouple
K—Warm Junction of Thermocouple
L—Freezing Cabinet
M—Motor
N—Temperature Control
P—Source of Power
R—Reversible Switch
S—Shunt Box
T—Thermograph
W—Condensing Coils
Y—Rubber Shoe
1, 2—Test Specimens

Figure 1.—Apparatus Used in Freezing Soils

as satisfactory results were obtained with them and a larger number of simultaneous tests could be run than with the larger containers. In experiments where it was desired to reduce friction between frozen soil and container, a lining of celluloid or of heavy greased paper was inserted. The cartons were strengthened when necessary by winding with tape saturated with shellac. The strongest containers used were made from heavy fiber cylinders 9 centimeters in diameter, having walls 7 millimeters thick. They were made impervious with paraffin or shellac, and, in tests conducted under the highest pressures, they were surrounded with wood strips, held in place by steel bands. Metal containers are unsatisfactory because of their rapid conduction of heat. It is also desirable that the containers be opened quickly before melting occurs.

In some tests it is necessary to maintain the water content of the unfrozen soil in the lower part of the carton, replacing it as fast as it is withdrawn because of freezing above, thus simulating the way in which water may be fed up from the water table during the freezing of soils in the ground. To accomplish this, small holes are punched in the bottom of the carton and covered with filter paper before introducing th soil. The carton is then placed in a vessel containin about one-half inch of coarse sand, saturated wit water. In saturating the side walls of the container with paraffin, care must be taken to keep the botto free of paraffin, otherwise water will not readily ente through the small holes. A collar, fitting snugl around the carton and resting on the top of the vesse prevents entrance of the dry sand in which the appa ratus is packed. The water level in the saturated san is maintained by adding water through a rubber tub which passes through the packing to the outside. Th pressures exerted during freezing are measured b observing the compression of a heavy steel sprin which has previously been calibrated.

Simple apparatus for recording the amount and rat of surface uplift during the freezing of water in a open system is shown in Figure 3. A thick-walle container, filled with the soil to be tested, stands in vessel containing sand, which is kept saturated wit water. It is held rigidly in place by a strong frame. heavy lead disk resting on the soil in the containe carries a pen which traces a continuous record on th

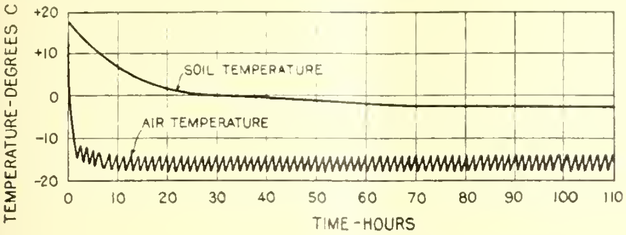
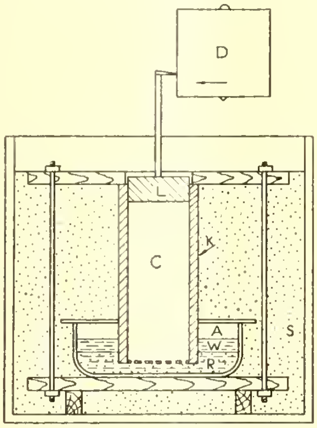


Figure 2.—Graphs Showing the Air Temperature in the Refrigerator and the Soil Temperature 12 Centimeters Below the Surface During a Freezing Experiment

revolving drum of a thermograph. As freezing gradually penetrates downward from the surface, the interstitial spaces in the soil below the zone of freezing furnish passages through which the water can move either upward or downward according to the forces to which it may be subjected, for it can easily enter or leave the container through the perforations in the bottom.



L—Lead Disk with Recording Pen
R—Sand Saturated with Water
D—Drum Turned by a Clock
K—Container
S—Dry Sand
W—Water
C—Soil
A—Air

Figure 3.—Apparatus for Freezing Soils in a System That is Open with Respect to Water. The Amount of Heaving is Recorded on a Rotating Drum

FREEZING PRODUCES DIFFERENT EFFECTS IN CLOSED AND IN OPEN SYSTEMS

When water is cooled under atmospheric pressure it normally freezes at 0° C. or a little below, with an expansion in volume of about 9 per cent. Increase in pressure, so long as it does not exceed 2,115 kilograms per square centimeter, lowers the freezing point. If water is frozen in a confined space, the pressure developed depends upon the resistance that is offered to expansion, but there is a limit to the pressure that can be obtained. The maximum pressure obtainable through the freezing of water in a closed space is, according to Bridgman, 2,115 kilograms per square centimeter; for at this pressure and − 22° C. ice III, which is denser than water, begins to form.[4] Tammann[5] obtained a slightly higher figure but the difference is not great. In a closed system, freezing results in pressure because the water is so confined that it can not escape, and the amount of heave varies directly with the quantity of water frozen.

In experiments performed with the apparatus shown in Figure 3 on soil systems that are open with respect to water it was found that soils differ greatly in the amount and rate of heaving, and that even the same soil will behave differently under different conditions of freezing. Some soils, under the most favorable conditions and when saturated with water, freeze with no appreciable uplift of the surface, while others give uplifts ranging up to more than 60 per cent of the depth of freezing. An uplift of even 100 per cent may be obtained through the formation of a layer of ice at the surface when the soil is too warm to permit freezing within the soil voids. When no heaving occurs, or when the heaving is less than that which results from the expansion in volume of the water frozen, it is obvious that some of the water must be pushed downward through the soil voids by the growing ice crystals and expelled from the soil container. On the other hand, when the surface uplift exceeds that which may be attributed to change in volume, additional water must enter the container as a result of the freezing.

SOILS OUT OF DOORS GENERALLY BEHAVE AS OPEN SYSTEMS

Observations out of doors show that most soils when subjected to freezing under natural conditions usually behave as open systems rather than as closed systems. In some places frost heaving is too great to be explained by the expansion in volume of water present in voids and in other places it is too little.

The average soil seldom contains as much as 50 per cent water, but if all the water in such a soil were to freeze in situ the change in volume could cause an uplift of less than 5 per cent of the depth of freezing. Moreover, in most soils having a high water content a considerable percentage of the water does not freeze. The depth of freezing in the colder parts of the United States seldom exceeds 2 or 3 feet; yet a surface heaving of 6 inches is not uncommon and an uplift of "a couple of feet" has been reported. The maximum amount of frost heaving that occurs during cold winters is not known, as very few accurate records are available, most observers having measured the difference in uplift of points located close together rather than the amount of uplift with reference to a bench mark that has not been disturbed by frost.

Although frost heaving at some points is excessive, at others the freezing of soils may be accompanied by no appreciable uplift. Frequently there is great difference in the amount of heaving within relatively short distances. Along a paved highway in New Hampshire the differential heaving was as much as 6 inches within a distance of 50 feet.

In laboratory experiments excessive heaving is always accompanied by the segregation of some of the water to form layers or lenses of more or less pure ice (see figs. 14, 17, and 18); and similar layers of segregated ice have been found when excavations have been made in badly heaved ground. The lump of clay containing thick ice layers, shown in Figure 4, was obtained in the spring of 1929 by Mr. F. C. Lang, of the Minnesota Department of Highways, under a street in St. Peter, Minn., where the amount of heaving was said to be a "couple of feet." It is obvious that ice layers are not formed in soil by the freezing of water in situ.

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[4] P. W. Bridgman, Proc. Amer. Acad., Vol. XLVII 1912), pp. 441-558.

[5] Gustav Tammann, "The States of Aggregation," translated by R. F. Mehl, New York (1925), pp. 158-175.

The local segregation of water in soil to give excessive frost heaving is usually compensated for, under natural conditions, by the expansion of air imprisoned in the soil or by the entrance of additional air through pores and cracks. The partial expulsion of water from soil voids in an area where freezing results in little or no heaving is usually compensated for by the compression or expulsion of air either locally or under adjacent areas.

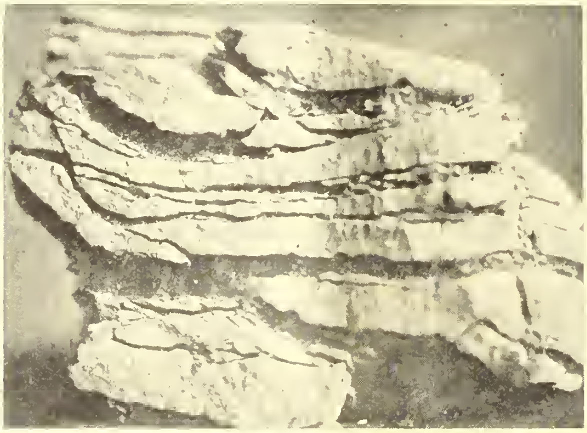

Figure 4.—Clay Containing Ice Layers from Under a Badly Heaved Street in St. Peter, Minn.

FEW SOILS BEHAVE AS CLOSED SYSTEMS

Probably few soils freezing naturally out of doors behave as absolutely closed systems with respect to water, for in order for this to occur the resistance to the movement of ground water has to be very high. In laboratory tests, soils that are very impermeable because of high colloid content behave essentially as closed systems, the amount of heaving being the same whether the containers are sealed at the bottom or whether they are perforated and stand in water. The surface uplift in both cases is equal to the change in volume of the water frozen. This subject is discussed under the heading "Tests made with soils containing bentonite." (See fig. 12, B.) Segregated ice forms in such soils when the water content is sufficiently high, but the ice crystals are built up of water drawn from the immediately adjacent soil. The only soils thus far tested that behave on freezing like closed systems, because of impermeability, are certain muck and gumbo soils, and soils containing bentonite.

If water begins to freeze simultaneously over a large area where air is excluded from the soil, the resistance to the movement of ground water may be so great as to give the effect of a closed system. Such conditions obtain where the water table is absolutely parallel to the surface and there is no local variation in soil or soil cover. These requirements practically limit closed soil systems to swampy areas; but even there, if the soil texture is favorable, lenses and layers of segregated ice may form. The uplift, however, will not exceed the volume change of the water frozen, except as voids formerly occupied by water become filled with air.

There is a gradation, and not a sharp demarcation, in the conditions that cause soils to freeze as open rather than as closed systems, and the conditions may change even during the freezing of soil. For example, when a sandy soil saturated with water begins to freeze it may behave practically as a closed system, the uplift equaling the change in volume of the water frozen; but as freezing progresses the weight of the frozen crust that must be lifted increases until the volume change is adjusted in part by forcing water out of soil voids immediately below; and finally, when the weight of the crust is sufficiently great, adjustment may be brought about almost entirely in this way, so that there is no uplift.

As resistance to the movement of ground water in soils increases an open system tends to grade into a closed system. Each individual area is a problem in itself, and it will take much field work as well as laboratory study to determine the relative importance of the different factors that determine whether a soil on freezing will behave as a closed or as an open system.

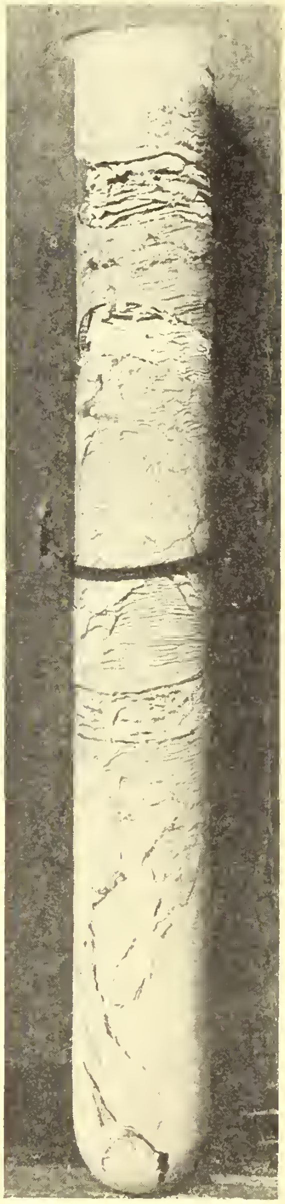

Figure 5.—Test Tube Containing Frozen Clay. Layers of Segregated Ice in the Upper Part of the Tube with Shrinkage Cracks Below

DIRECTION OF GROWTH OF ICE CRYSTALS AN IMPORTANT FACTOR

The pressures resulting from freezing are due to the growth of crystals. A crystal can develop pressure only in those directions in which it is growing. If the growing crystal exerts pressure against a liquid that is confined, hydrostatic pressure is the result; but in a closed system the crystal is able to exert pressure only if crystallization is accompanied by an increase in volume.

In open systems, where the liquid can escape, the pressures resulting from freezing are not hydrostatic, but are due directly to crystal growth, and are effective only in the direction of growth. Liquids such as benzene and nitrobenzene, that solidify with decrease in volume, give a pressure effect similar to that of water when substituted for it in freezing experiments with open systems, though they are incapable of developing pressure when frozen in closed systems.

An ice crystal grows in those directions in which it is in contact with undercooled water; or, stated in a different way, the growth of an ice crystal in any particular direction is determined by three factors: (1) The presence or absence of water in contact with the crystal in that direction, (2) the temperature at the contact, and (3) the pressure at the contact. In most cases where water freezes as a result of natural processes, heat is conducted away from the growing ice crystals in one direction and they are in contact with water only in the opposite direction; therefore growth takes place in a single direction. This is the way water freezes in lakes. The factors that control the direction of crystal growth are illustrated in the experiments described below.

It is difficult to break thin glass test tubes by freezing clear water in them if they are uniform in bore and open at the top, even though they be exposed to cooling from the side instead of from the top, because water expands on cooling from 4° to 0° C., and therefore the coldest water tends to rise and come in contact with the downward growing crystals. If the tubes are sealed, or if there are irregularities in the bore, hydrostatic pressure develops and rupture occurs at the weakest place. Sand placed in the water will prevent convection currents, but the test tubes are not usually ruptured on freezing, for the crystals stop growing horizontally when they come in contact with a solid.

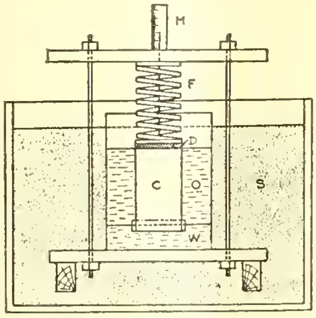

Figure 6.—Apparatus for Freezing Clay Cylinders Under Heavy Vertical Pressure with no Lateral Support

If clay or some other very fine material is introduced in the water, convection currents are prevented, but, what is more important, water is able to reach the surfaces of growing ice crystals through the interstitial capillary spaces. This is due to the fact that water does not readily freeze in very small capillaries.[6] Test tubes containing clay and water, when exposed to cooling from the sides, are broken, for the ice crystals grow radially inward and exert pressure against the walls of the tubes. Similar tubes, buried in dry sand so that cooling is from the top down, are not broken, for the crystals grow in a vertical direction. Alternating layers of clear ice and frozen clay are formed near the top of the tubes, while shrinkage cracks develop in the clay below as water is withdrawn to build the ice layers above. The test tube shown in Figure 5 was slightly cracked, accidentally, before it was buried in sand. On freezing, the cracks extended around the test tube and the fragments were separated in a vertical direction, but no longitudinal cracks appeared, for the crystals did not grow from the wall inward.

Similar freezing tests have been made on open systems where it was possible for water to enter or be expelled. A cylinder 6 centimeters in diameter, cut from somewhat indurated clay, was placed in a small tray with perforated bottom, and moist clay tamped around it to hold it in place, a strip of adhesive tape having been wrapped around the bottom of the cylinder to help prevent slaking because of rapid absorption of water. The tray was embedded in a layer of sand saturated with water contained in a large carton (see fig. 6), and then oil was poured in until level with the top of the clay. This oil is sufficiently viscous when cold to eliminate convection currents, while it does not solidify at low temperatures. A steel disk placed on the clay cylinder supported a stiff spring which could be compressed against a plate held in place by bolts. The vertical pressure and the amount of heaving were shown on a scale by the position of an indicator that extended from the clay up through the spring. The apparatus was buried in dry sand so that freezing was from the top down.

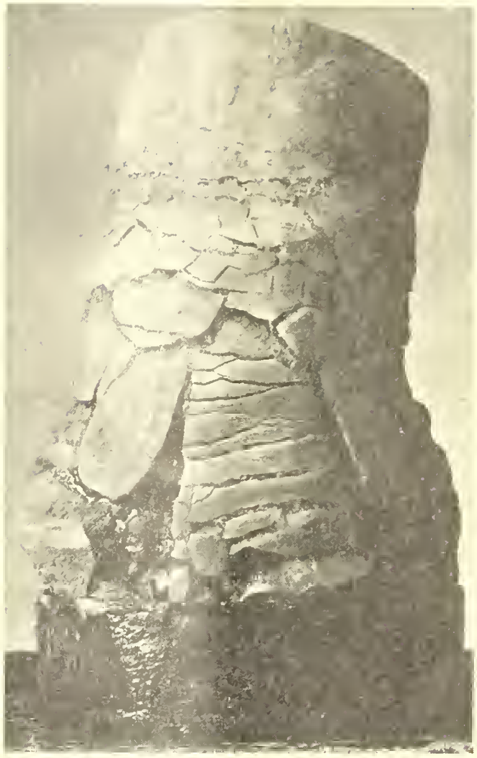

Figure 7.—Clay Cylinder Frozen Under Heavy Vertical Pressure with no Lateral Support

In one test the bolts were tightened to give an initial pressure of 101 pounds before cooling was started. Five hours later the pressure had dropped to 94 pounds because of slow failure of the cylinder, but after 15 hours heaving had restored the initial pressure, and at the end of 76 hours, when freezing had reached the bottom, the pressure was 140 pounds (2.3 tons per square foot). The heaving was due to the formation of horizontal layers of fibrous ice ranging up to 0.5 centimeter in thickness. (See fig. 7.) The cracks, due to failure of the cylinder under pressure, contained almost no ice, being filled chiefly with oil. In this test heaving was upward in spite of heavy vertical pressure and practically no lateral resistance. In tests conducted under less initial pressure, failure of the clay cylinders did not occur, and their appearance after freezing was similar to that of the cylinders shown in Figures 17 and 18. The pressure developed in this experiment was limited by the crushing strength of the clay cylinder.

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[6] Stephen Taber, "The Growth of Crystals Under External Pressure," Am. Jour. Sci., Vol. XLI (1916), pp. 544-545, 552.

In another experiment clay was packed around copper bars in a carton with perforated bottom which stood in sand kept saturated with water. The apparatus was then buried in dry sand and subjected to freezing temperatures. Resistance was least in a vertical direction, but rapid conduction of heat by the copper caused ice crystals to grow outward from its surface (see fig. 8), and the carton was ruptured near the bottom.

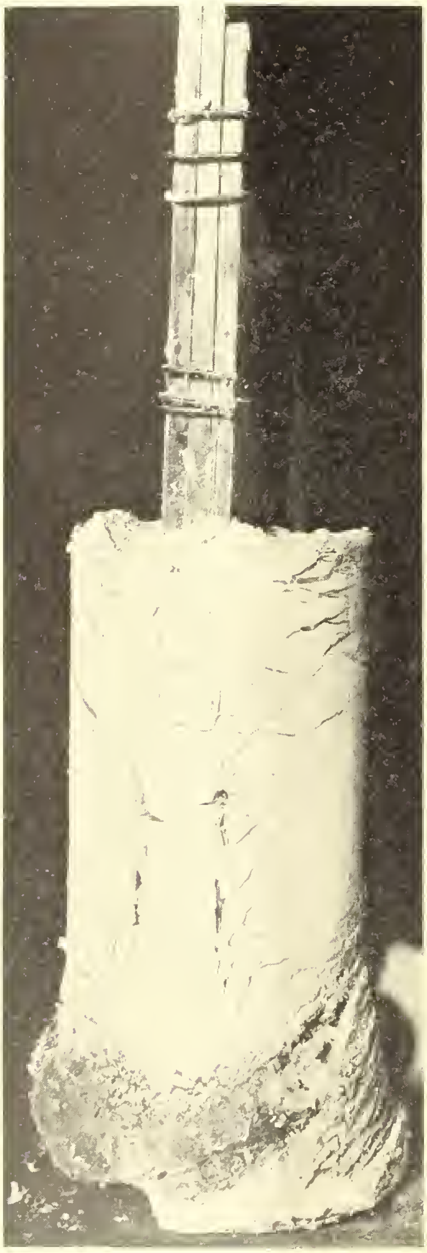

Figure 8.—Clay Cylinder Enlarged by Growth of Ice Crystals Outward from Copper Bars

Wet clay was packed in wrought-iron cylinders having a bursting strength of over 11,000 pounds per square inch, the ends being sealed with wooden plugs. On its being subjected to freezing temperatures, ice crystals grew radially inward at first, forming a layer of ice in contact with the metal, but as the pressure increased the end plugs were slowly driven out by the growth of ice crystals in the direction of least resistance. These crystals built up ice layers transverse to the axis of the cylinder. The ice crystals grew in the direction of least resistance, although heat conduction was most rapid in a transverse direction. However, the difference in temperature in different directions must have been small. Ice crystals will continue to grow in those directions in which they are in contact with water, if the temperature at the ice-water surface is below the freezing point for the existing pressure. With any increase in pressure the temperature must be lowered if crystal growth is to continue, but a pressure of 145 kilograms per square centimeter is required to lower the freezing point from 0° to −1° C. Effects that may be attributed to difference in pressure or difference in the resistance to crystal growth are rarely observed where soils freeze under natural conditions; and the direction of heaving is, therefore, usually determined by the direction of heat conduction and the availability of water.

VARIOUS FACTORS DETERMINE ICE SEGREGATION IN SOILS

The physical laws that control the freezing of water are the same in the laboratory as out of doors, but in the laboratory it is possible to vary one factor at a time while the others are kept constant, a method that is not usually possible under natural conditions. This is especially true of the factors that control ice segregation in soils; and these factors are of major importance, since they determine the amount of heaving in open systems. The effects of different factors on the freezing of water in soils were described in the paper on Frost Heaving, from which the following account is in part abstracted. Subsequent study has disclosed some additional facts which are included.

TEXTURE OF SOILS AFFECTS ICE SEGREGATION

Texture, as applied to soils, refers to the size, shape, and arrangement of the individual particles, the degree of consolidation, and, largely determined by the foregoing, the size and percentage of voids.

Size of soil particles is one of the most important factors controlling segregation of water during freezing. As the maximum size permitting segregation varies with the other factors, it is essential that all tests be carried through under exactly the same conditions if comparable results are to be obtained.

Quartz ground to pass a 200-mesh sieve (maximum particle diameter about 0.07 millimeter) showed faint segregation near the surface under favorable conditions of cooling and no surface load. (See fig. 9, A.) The material was packed in a carton which was buried to the top in dry sand while the bottom was kept saturated with water. The initial temperature was 20° C. The air temperature at the surface was then rapidly lowered to −15° C. (see fig. 2.) and maintained with a variation of only about 3° until the close of the test. There was no surface load, but as freezing progressed downward the weight of material that had to be lifted gradually increased as did likewise the friction of the frozen soil against the walls of the carton. Quartz dust having an average particle diameter of between 6 and 10 microns gave slightly greater evi-

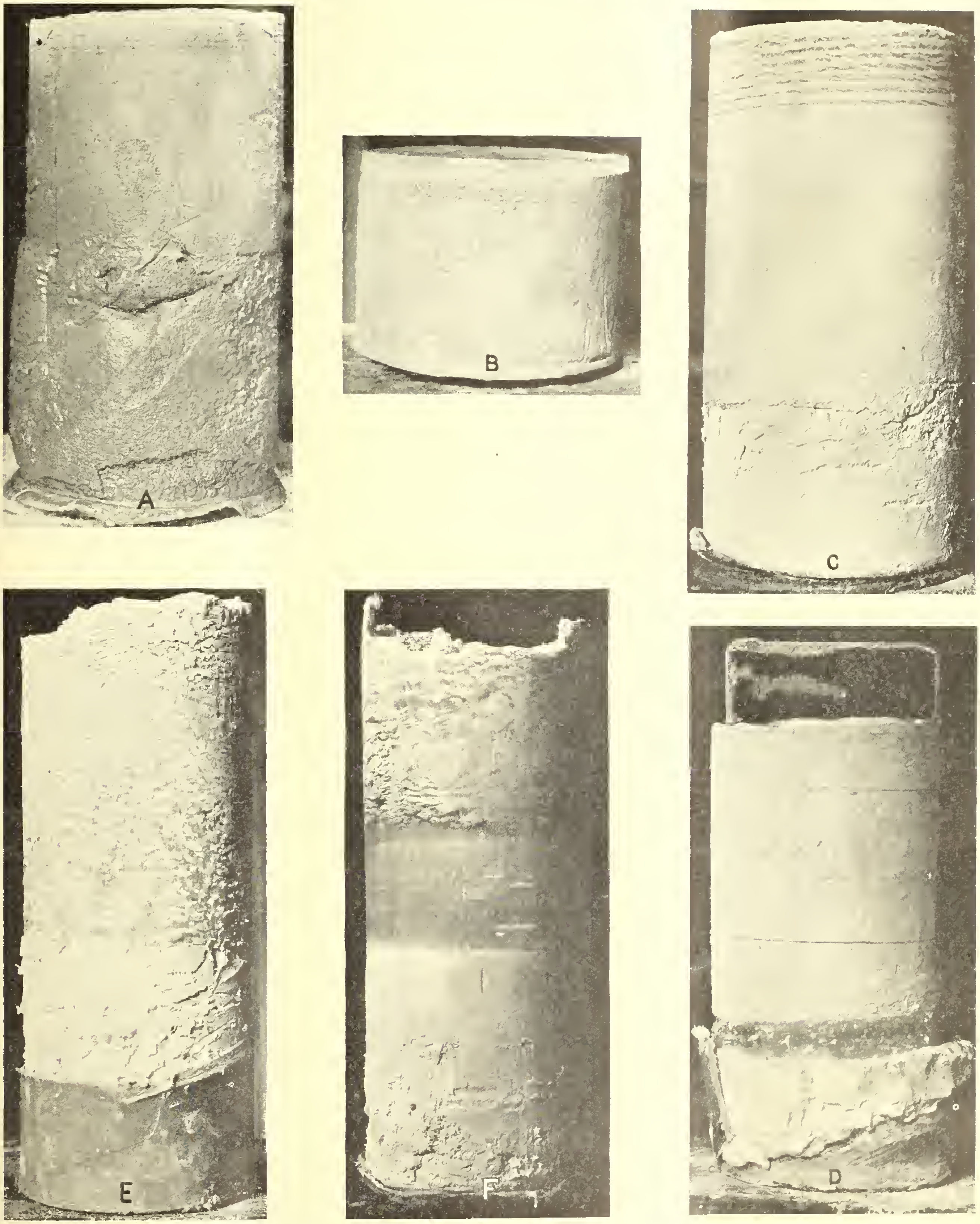

Figure 9.—Specimens Showing Influence of Soil Texture on Formation of Segregated Ice: A—Quartz Flour, Maximum Particle Size About 0.07 Millimeter, Showing Slight Banding near Top Due to Ice Segregation. B—Quartz Flour, Particle Size 6 to 10 Microns. Slight Banding Due to Ice Segregation. C—Barium Sulphate, Particle Size About 2 Microns, Showing Segregation of Ice. D—Barium Sulphate with Ice Layer at Lower Limit of Freezing; Thin Ice Layers Above. Iron Weight on Top. E—Lithopone, Particle Size One-half Micron, Showing Segregated Ice. F—Kadox, Particle Size One-fourth Micron, with Layer of Pure Ice 5 Centimeters in Thickness.

dence of segregation when tested in the same way. (See fig. 9, B.)

In some of the tests chemically precipitated crystals of various substances were substituted for quartz in order to obtain greater uniformity in particle size. Barium sulphate, having an average particle diameter of about 2 microns and a maximum diameter of only 3 microns, gave well-defined segregation when tested in the manner described above (see figs. 9, C and 9, D), but under a surface load of 30 pounds per square inch no heaving occurred; and, also, when the material as a whole was brought to a temperature of 0° C., before freezing rapidly from the top down, no segregation took place.

Segregated ice forms easily in materials with a particle size of 1 micron or less. Results obtained with lithopone, having a particle diameter of about one-half micron, and with kadox, which has a probable particle size of one-fourth micron or less, are shown in Figures 9, E and 9, F. The clay used in most of the tests is a very pure South Carolina Cretaceous white clay, which consists largely of particles having apparent diameters of between one-half and one micron; but it is very difficult to distinguish the ultimate particles from aggregates. Part of the material is much finer.

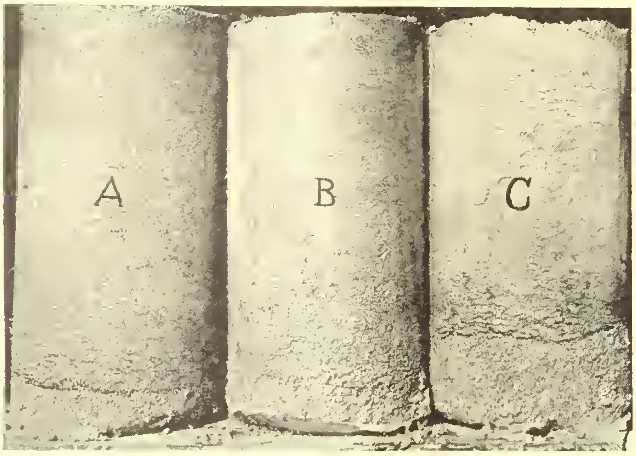

Figure 10.—Frozen Cylinders Containing (A) 20 Per Cent, (B) 25 Per Cent, and (C) 30 Per Cent Clay Mixed with Sand

LARGE SOIL PARTICLES PREVENT ICE SEGREGATION

The effect of large soil particles in preventing ice segregation is illustrated by the following experiments with mixtures of pure clay and standard Ottawa sand. This sand is very uniform, the grains having an average diameter of about 0.7 millimeter. Mixtures containing slightly less than 20 per cent clay by weight would have just enough clay to fill the voids between the sand grains so that the size of the pore spaces would be the same as in pure clay. On freezing no segregated ice could be observed in mixtures containing less than 30 per cent clay, and even with 50 per cent clay the amount of segregation is small as compared with tests made on pure clay. The results obtained by freezing sand-clay mixtures are shown in Figures 10 and 11.

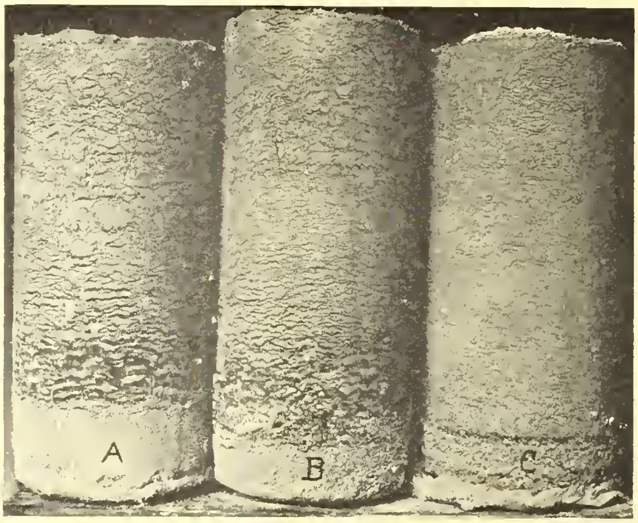

Figure 11.—Frozen Cylinders Containing (A) 50 Per cent, (B) 40 Per Cent, and (C) 30 Per Cent Clay Mixed with Sand

Under more favorable conditions than those prevailing in the tests described above, segregated ice will form in soils having a somewhat larger particle size, whether the soils be of uniform grain size or contain appreciable amounts of coarser particles. In these laboratory tests freezing progresses more rapidly near the beginning than it usually does under natural conditions, and, after freezing has penetrated downward far enough to slow up the process, heaving is resisted by friction between frozen soil and container. The effects of rapid freezing and of resistance to heaving in preventing the formation of segregated ice are discussed later in this paper.

TESTS MADE WITH SOILS CONTAINING BENTONITE

Experiments were also conducted on soils containing different percentages of colloidal material. Colloidal particles are supposed to range in size from about 0.1 to 0.001 of a micron, but the size of particle in the colloidal materials used is not known. Most of the tests were made on bentonite and mixtures of bentonite with the South Carolina Cretaceous white clay. The bentonite used consists chiefly of minute micaceous crystals of montmorillonite or beidellite, which separate into flakes of colloidal thickness, while the Cretaceous clay is, for a clay, relatively low in colloidal material.

When mixed with water, bentonite expands to several times its original volume, and when wet it becomes very impermeable. Cartons packed with dry bentonite and stood in wet sand are ruptured by the change in volume. When packed in a strong glass container having a perforated bottom, water was absorbed rapidly through the bottom until a thin layer of bentonite became saturated, then the entrance of water became extremely slow, and at the end of nine months it had penetrated to a height of only 4 centimeters, even though the water was kept under a head of 30 centimeters. It was therefore necessary in freezing tests to mix the bentonite thoroughly with water before placing it in cartons.

Mixed with water bentonite forms a jellylike mass, which is so impermeable that water can be drawn only a short distance to build segregated ice. On freezing from the top downward, tension is set up in the bentonite immediately below a growing ice layer. Since the stress is uniformly distributed and the material homogeneous, vertical cracks tend to develop, which in horizontal sections form a polygonal pattern. (See

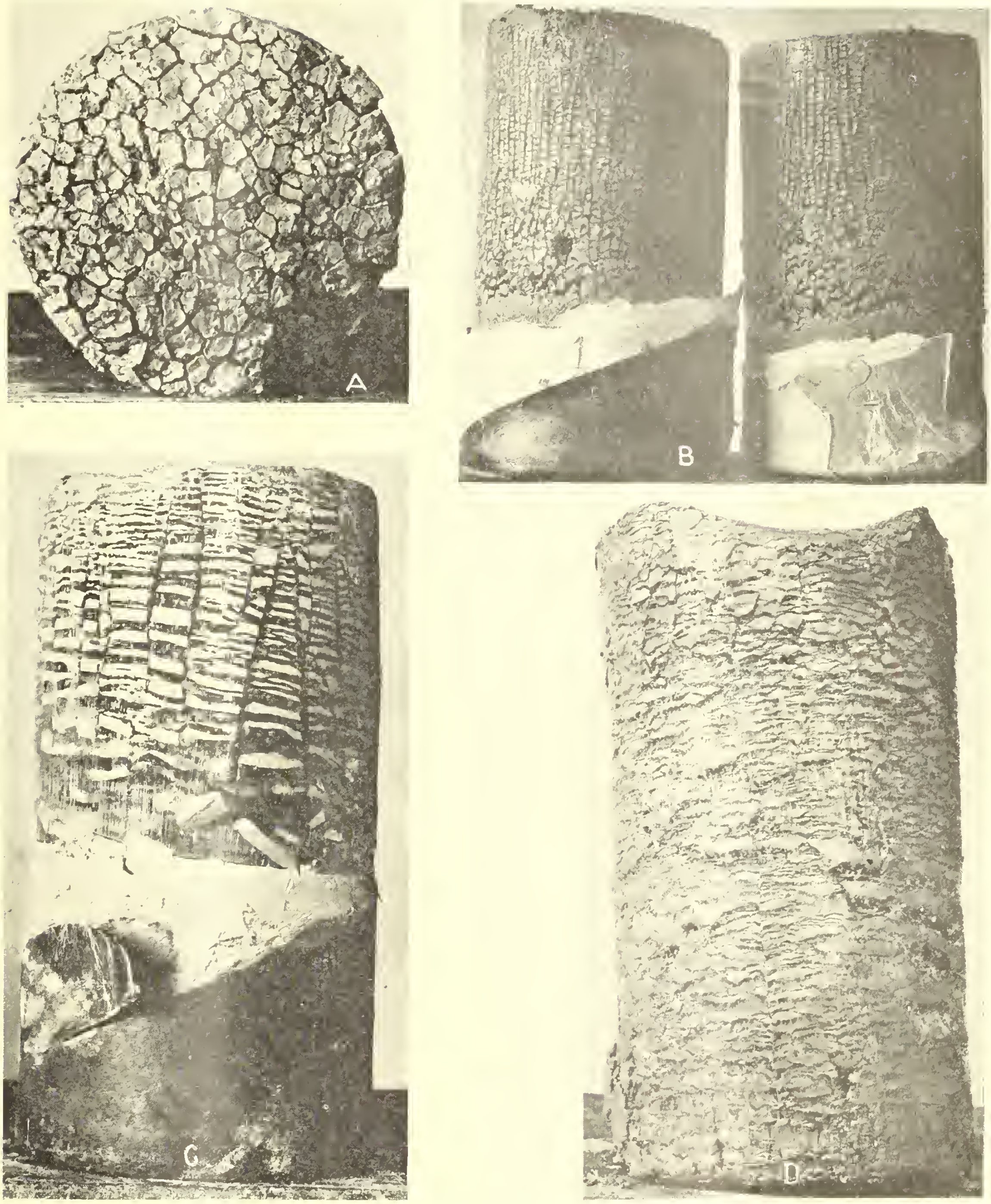

Figure 12.—Results of Freezing Tests Made on Specimens Containing Bentonite. A.—Polygonal Pattern Formed on Freezing a Mixture Containing 25 Per Cent Bentonite and 75 Per Cent Clay. B.—Cellular Structure Formed on Freezing a Mixture of Bentonite and Water. Carton No. 1 Stood in Water. Carton No. 2 Was Sealed at the Bottom. C.—Cellular Structure Formed on Freezing 94 Per Cent Clay and 6 Per Cent Bentonite Mixed with Water. D.—The Result of Freezing 4 Per Cent Bentonite Mixed with 96 Per Cent Clay While Standing in Wet Sand

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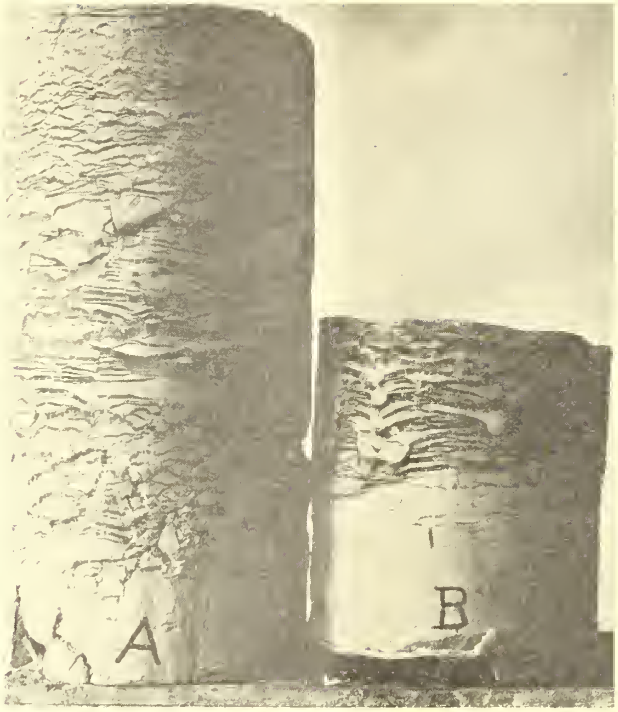

	Clay columns	
	A	B
Diameter of clay column.......centimeters..	8. 5	8. 5
Height of surface above water table.......do....	20. 1	5. 5
Height of clay column before freezing.......do....	23. 5	7. 7
Height of clay column after freezing.......do....	29. 9	9. 5
Amount of uplift.......do....	6. 4	1. 8
Time in freezing box.......days..	5	5

FIGURE 13.—CLAY CYLINDERS FROZEN WITH LOW WATER TABLE (A) AND HIGH WATER TABLE (B). LOWER HALF OF A REMOVED

fig. 12, A.) The cracks are gradually filled with clear ice, and as freezing progresses they advance downward, forming a columnar structure, which, combined with the normal horizontal ice layers, results in the peculiar cellular structure shown in Figures 12, B and 12, C.

The results of freezing pure bentonite mixed with 83 per cent water are shown in Figure 12, B. Carton No. 1 stood in sand saturated with water, while carton No. 2 was sealed at the bottom. No water entered the impervious mixture in carton No. 1, for the weight did not change appreciably during the experiment. Mixing clay with the bentonite so as to decrease the amount of colloids causes the polygonal cracks to become more widely spaced. Figure 12, C shows the result of freezing a mixture containing 6 per cent bentonite and 94 per cent clay in a carton that was closed at the bottom.

A dry mixture of clay with 4 per cent bentonite was sufficiently permeable so that, when the container was placed in saturated sand, water was slowly absorbed. The result of freezing this mixture as an open system is shown in Figure 12, D.

Tests were run on several natural soils which in open systems gave little or no heaving beyond that due to change in volume of water frozen. These soils contained a relatively high percentage of colloids, for they swelled greatly on absorbing water, and were so impermeable that water could not rise in the dry material higher than 10 or 15 centimeters in a period of a week. Some of these soils were clays of the gumbo type and others were muck soils containing much partly decomposed vegetable matter.

The clay minerals occur chiefly in the form of flat tabular particles, which therefore have a larger surface than particles of the same volume that are more nearly spheroidal. Also, when the particles are given parallel orientation the voids are smaller in diameter than in soils made up of irregular or spheroidal particles having the same volume.

The size of the capillary spaces in soils determines the height to which water may be lifted above the water table by surface tension, the height varying inversely as the diameter of the capillaries. In very fine soils water may rise 3 meters or more, but most fine soils contain colloids which decrease permeability and retard the movement of water.

During the growth of an ice layer in soil, water is not supplied by capillarity, for there is no free surface or meniscus. The force causing the upward flow of water to feed growing ice crystals is greater than that which results in the capillary rise of water in soil. This fact is illustrated in the following experiment.

Powdered clay was tightly packed in two cartons, of equal diameter but different height, which stood in sand saturated with water. The clay in the short carton became saturated with water in about 24 hours, but seven days passed before the clay in the tall carton became slightly moist on top although it was subjected to frequent tamping.

On freezing, however, water was drawn up to form ice layers in the tall carton almost as easily as in the short one. (See fig. 13.) Before making the photograph the lower half of the tall clay column was removed. The vessel in which the tall carton stood contained the most water, but in both cases all of the available water was withdrawn to form ice layers; and shrinkage cracks developed in the lower half of the tall clay column.

A thin layer of coarse sand will prevent the upward movement of water during freezing experiments. This action is probably due to interruption of the water supply by the accumulation of air in the large voids in sand.

Water occupying very small voids in soil does not freeze readily and may be undercooled in the immediate vicinity of ice crystals. When a cylinder of partly frozen clay, such as is shown in Figure 14, A, is tested with the point of a knife, the clay between the lowest ice layers is found to be soft as compared with that higher up, the hardness increasing toward the top, where the temperature was lower and freezing had gone on for a longer time.

At a given depth ice probably forms in the larger voids of clay first, and then, freezing gradually, extends to the smaller voids; but careful tests with a specially devised dilatometer show that part of the water does not freeze on prolonged exposure to low temperature. In the Cretaceous white clay 6 grams of water per 100 grams of dry clay do not freeze. Similar results were obtained with large fragments of porous porcelain saturated with water, thus proving that pore space, or the amount of surface exposed, is the essential factor rather than size of particle. In coarse sand or other material having a relative small area of surface all of the water appears to freeze. The water remaining unfrozen in fine soils is probably adsorbed water, for adsorption is a surface phenomenon.

The degree of consolidation is an important factor in determining the amount and rate of heaving of soils. A cylinder of undisturbed Cretaceous clay will heave at the rate of 0.15 millimeter per hour, while the same clay when pulverized and tightly packed will heave 0.8 millimeter per hour. The difference in the amount of segregated ice formed is clearly shown in the photographs. (See figs. 14, A and 14, B.) The cylinder of undisturbed clay was frozen to a greater depth, for it remained in the refrigerator for seven days, while the pulverized clay heaved so rapidly that it came in contact with the support of the recording drum and had to be removed after only four days. Otherwise, both tests were frozen under the same conditions.

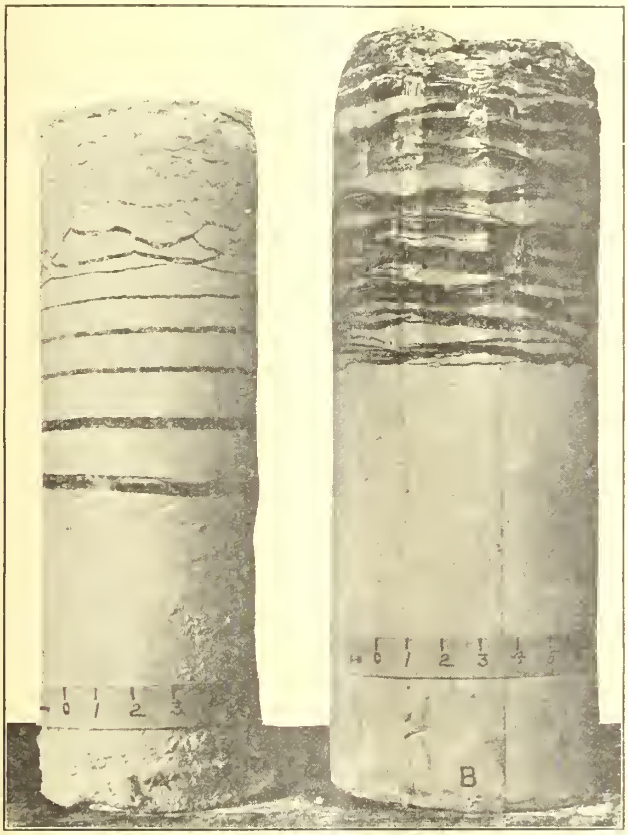

Figure 14.—Tests Showing Influence of Degree of Consolidation on the Amount and Rate of Heaving of Soils. A.—Cylinder Cut from Undisturbed Cretaceous Clay and Frozen While Standing in Wet Sand. The Surface Uplift Equals the Total Thickness of the Ice Layers. B.—Pulverized Clay, Tightly Packed and Frozen While Standing in Wet Sand

The difference in the behavior of partly compacted and thoroughly compacted clay is due to several causes. The most important is probably difference in permeability. In consolidated clay some of the particles are probably in such close aggregation that they conduct heat as though the clay were partly composed of larger particles. Also, the tensile strength of the undisturbed clay is much higher.

COMPOSITION OF SOILS AN IMPORTANT FACTOR

The most abundant minerals in soils are quartz, feldspar, and the kaolin minerals. Quartz and feldspar occur as irregular grains; the kaolin minerals are usually much smaller in particle size, and some of them, with micaceous cleavage, separate readily into flakes of colloidal thickness. Humus is rarely an important constituent of subgrade soils, but locally it may make up the bulk of a soil. Colloids left by the decay of organic matter, as well as mineral colloids, decrease the permeability of soils.

The thermal properties of a soil depend upon the conductivities and specific heats of the constituents. Because of lower conductivity and higher specific heat humus changes temperature more slowly than minerals. The soil minerals differ little from one another in their thermal properties but they differ greatly from water. Since minerals cool more rapidly than water, mineral particles, and especially large ones, favor the downward movement of the freezing isotherm in soils, and therefore tend to check the growth of layers of segregated ice.

Other things being equal, soils with a high percentage of organic material might be expected to favor ice segregation, but from the tests so far made the difference appears to be slight. Most of the soils of this type so far tested are muck soils containing a high percentage of clay and so much colloidal material as to make them very impervious.

WATER SUPPLY INFLUENCES ICE SEGREGATION

When soil freezes as a closed system the only water available is the interstitial water in the zone of freezing; therefore the amount of heaving is determined by the change in volume of this water. When soil freezes as an open system and segregated ice forms, additional water may be drawn up from the water table if the soil is saturated from the zone of freezing downward, or it may be derived from the suspended subsurface water retained against the force of gravity above the capillary fringe.

Heaving is apt to be excessive at points where the water table is locally so close to the surface that the capillary fringe extends up into the zone of freezing, but essentially similar conditions obtain if freezing occurs when the soil is temporarily saturated from the freezing zone down to the capillary fringe with downward percolating water from recent rains or from melting snow or ice. If thawing is from the surface downward in the spring the water set free can not escape by downward percolation until after most of the ice has melted. Impermeable frozen soil may therefore establish a temporary, perched water table, and the excessive amount of water thus retained at the surface is available for building up masses of segregated ice in case temperature drops below freezing. Experiments show that the water content of a well consolidated soil does not change while water is being drawn through it from the water table below to build ice layers in the zone of freezing above.

When segregated ice is formed from water retained in the soil above the capillary fringe, the withdrawal of water from one point to form ice at another may at first result in shrinkage, but if the process continues air will enter or expand to fill the voids vacated by water. In freezing tests on samples of white clay containing from 9.5 to 29 per cent water[7] the percentage of the water present that was withdrawn from the lower part of the clay and concentrated in the frozen upper part tended to increase slightly as the amount of water

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[7] The percentage of water in soils as used throughout this paper is on the basis of the weight of the wet soil.

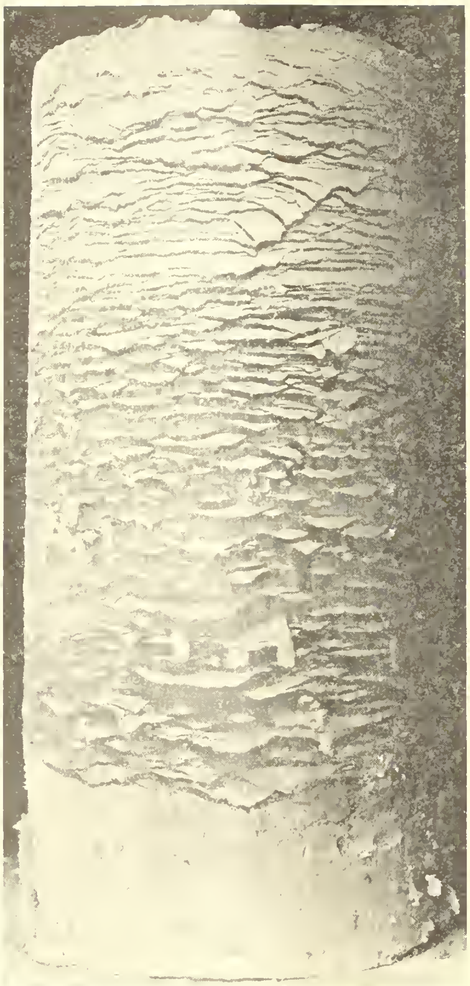


Figure 15.—Clay Cylinder Frozen Under 30 Pounds of Iron Shows Excessive Segregation of Ice

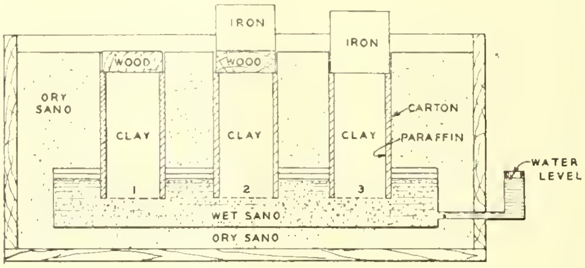
in the soil increased; but even in clays with least water some segregation took place. The porosity of the undisturbed, and therefore thoroughly consolidated, clay used in these experiments is 44 per cent, and when the clay contains about 23 per cent water all of the voids are filled.

REMOVAL OF HEAT DETERMINES EXTENT OF ICE SEGREGATION

When a particular soil having a maximum particle diameter of less than 1 micron is frozen under little or no load in an open system, the amount of segregated ice formed is determined chiefly by the amount by which the heat content of the soil is lowered; and this depends on several factors, such as the length of the cold spell, the minimum temperature reached, the heat present in the soil, and the amount of heat reaching the surface soil from greater depth. The rate at which heat is removed from a soil during freezing, and therefore the rate of heaving, varies according to the conductivity and specific heat of the soil cover.

In one freezing test, iron weights were piled on tightly packed clay to give a pressure of 0.5 kilogram per square centimeter, but, in spite of the pressure, heaving was faster than in similar tests under no load, because of the rapid conduction of heat by the iron. The uplift was 6.4 centimeters and the depth of freezing 15 centimeters after 114 hours in the freezing box. The large amount of segregated ice formed is shown in Figure 15.

In a series of tests carried out to determine the effect of different kinds of soil cover, cylinders of the same size cut from undisturbed clay were used, as greater uniformity could be assured in this way than by packing the pulverized clay. Three of these cylinders were placed in cartons with perforated bottoms and surrounded by paraffin. The cartons stood in sand kept saturated with water as shown in Figure 16. A disk of wood weighing 0.1 kilogram rested on one cylinder, a similar disk of wood and iron weight totaling 3.1 kilograms rested on the second cylinder, while the third was loaded with 3.1 kilograms of iron. The depth of freezing and the ratio of uplift to depth of freezing were both greatest where iron, a good conductor of heat, was in contact with the clay, and were least where wood, which served as a heat insulator, was the only covering. Complete data for the experiment are tabulated under Figure 17, which shows the clay cylinders at the close of the experiment.



CYLINDER NUMBER	1	2	3
LOAD (GMS. PER SQ. CM)	2	60	60
DEPTH OF FREEZING (CM)	8.5	10.0	10.8
UPLIFT (CM)	3.2	4.1	4.5
RATIO OF UPLIFT TO DEPTH OF FREEZING	0.38	0.40	0.41
TIME IN FREEZING BOX (HRS.)	74	74	74
INITIAL TEMP. OF AIR (C)	+16	+16	+16
FINAL TEMP. OF AIR (C)	−14	−14	−14

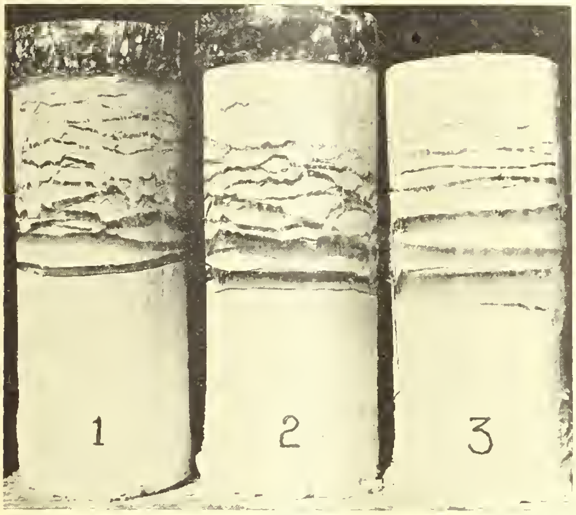
Figure 16.—Apparatus for Freezing Clay Cylinders When Weighted with Different Materials

Similar tests were performed on three clay cylinders, one of which supported 7.4 kilograms of iron, while another with the same load had a wooden disk between the clay and iron, and the third had nothing on it. The depth of freezing and ratio of uplift to depth of freezing were again greatest where the iron rested on clay, and they were least where the wood retarded the removal of heat from the clay. (See fig. 18.)

Under moderate loads, therefore, pressure has less influence on the heaving of clay than the thermal properties of the material used in applying the load.

In the six tests described above the ratio of uplift to depth of freezing appears to vary with the rate of heat removal, but the differences in the ratios were not great, and other tests indicate that rapid freezing tends to reduce the ratio of uplift to depth of freezing rather than increase it. These anomalous results were probably due to the fact that less work was required in drawing water up to the growing ice layers in tests where freezing approached close to the water table.

In open systems where heaving is largely if not entirely due to the formation of segregated ice the most favorable places for the development of ice layers are close to the surface and near the lower limit of frost penetration—points where the freezing isotherm descends the slowest. Ice needles develop at the surface of moist clayey soils when the temperature of the ground immediately below the surface remains above freezing, while the air temperature is below freezing; they do not form when previously chilled soils are rapidly frozen during a sudden drop in temperature. Under similar conditions thin layers of ice form under stones and pavements.



	Cylinder No.		
	1	2	3
Height of cylinder before freezing ....centimeters..	14.3	14.3	14.3
Diameter of cylinder ....do....	7.8	7.8	7.8
Load, grams per square centimeter	2.0	60.0	60.0
Depth of freezing ....centimeters..	8.5	10.0	10.8
Uplift ....do....	3.2	4.1	4.5
Ratio of uplift to depth of freezing	0.38	0.40	0.41
Time in freezing box ....hours..	74.0	74.0	74.0
Initial temperature of air ....° C..	+16°	+16°	+16°
Final temperature of air ....do....	−14°	−14°	−14°

FIGURE 17.—CLAY CYLINDERS FROZEN UNDER LOADS AS SHOWN IN FIGURE 16

In material barely fine enough to permit segregation, such as the barium sulphate previously described, well-defined layers of ice may develop only near the top and at the bottom of the frozen material. (See figs. 9, C and 9, D.)

The absence of well-defined ice layers near the top of the test specimen shown in Figure 9, D was due to rapid conduction of heat by a 4-pound iron weight that rested on it during the experiment. The formation of ice layers was entirely prevented by packing the carton of barium sulphate, together with the wet sand in which it stood, in ice for 18 hours to bring the system to 0° C., and then transferring it quickly to the sand box, which had been brought to approximately the same temperature. Under these conditions the freezing isotherm moved rapidly downward as the air temperature in the refrigerator was about −15° C. Finer material, when treated in this way, was not appreciably affected, and ice layers developed about as usual.

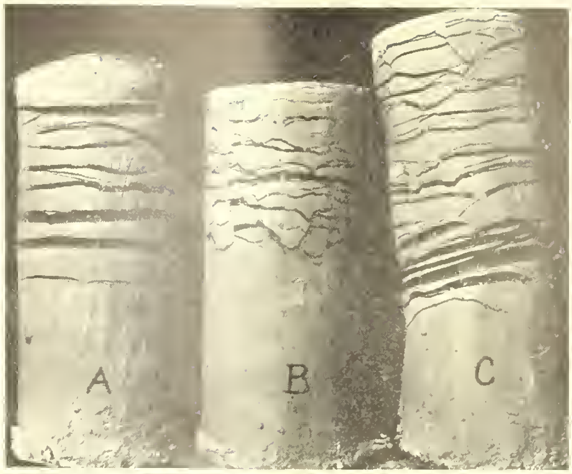


FIGURE 18.—CLAY CYLINDERS FROZEN (A) UNDER NO SURFACE LOAD, (B) UNDER IRON WEIGHT INSULATED FROM CLAY BY WOODEN DISK, AND (C) UNDER IRON WEIGHT IN CONTACT WITH CLAY

SURFACE LOAD AFFECTS ICE SEGREGATION

A relatively small surface load will entirely prevent frost heaving in an open system if the material is of such texture that only a little segregated ice forms under the most favorable conditions. A pressure of 2.11 kilograms per square centimeter prevented ice segregation and frost heaving in the barium sulphate (particle diameter 2 microns), but similar pressures resulted in very little reduction of heaving in kadox (particle diameter one-fourth micron) or in the pure white clay.

A cylinder cut from undisturbed clay and frozen under a surface load of 3.52 kilograms per square centimeter is shown in Figure 19. In some of the tests on silts and gritty clays a little heaving occurred under no surface load while material within an inch or two of the surface was being frozen, but as freezing progressed downward the friction of the frozen soil against the walls of the container was sufficient to stop ice segregation and therefore heaving.

Apparatus devised to measure pressures developed during the freezing of soil in an open system is shown in Figure 20. A carton containing the soil stands in sand kept saturated with water, and a spring, resting on top of the soil, is compressed against a steel plate as heaving takes place, the amount of uplift and the pressure developed being indicated on a scale by a wire pointer that extends up through the spring. The carton shown in the photograph had been strengthened with tape saturated with shellac. The wooden boards fit snugly around the carton and hold it rigidly in place.

The results of a test made on tightly packed clay are shown in Figure 21. The carton stretched somewhat under the high pressure both above and below the board that surrounded it, and this explains the more or less vertical ice veins. The maximum pressure measured was 7.31 kilograms per square centimeter, but this figure does not take into account the frictional resistance of the frozen clay in contact with the container, which must have been high.

In a test with a cylinder 13 centimeters high and 5 centimeters in diameter cut from undisturbed clay and

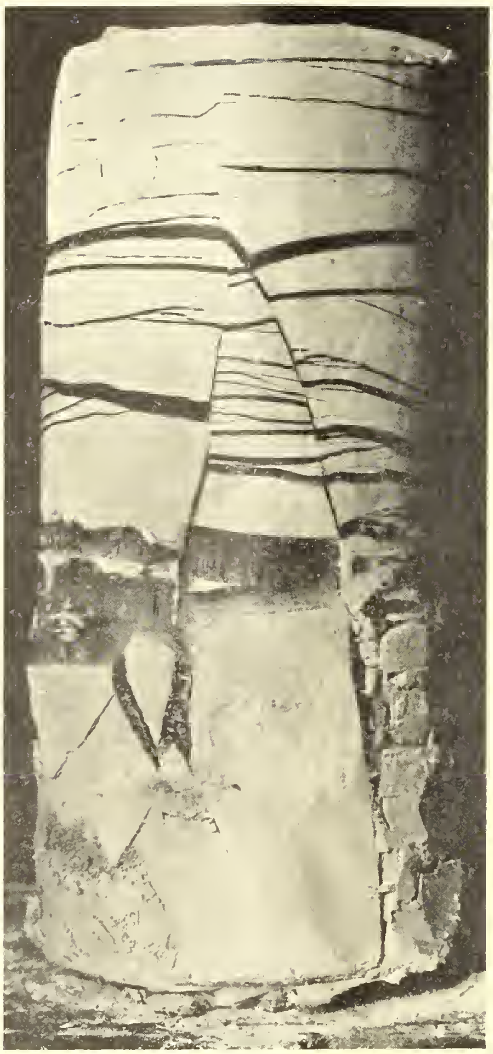


Figure 19.—Clay Cylinder Frozen Under a Load of 3.52 Kilograms Per Square Centimeter While Standing in Wet Sand

surrounded with paraffin, a pressure of 11.25 kilograms per square centimeter was obtained. The layers of segregated ice were few and small, the maximum thickness being about 1 millimeter. In tests performed under slightly higher pressure heaving was almost imperceptible.

Only rough estimates can be made of the maximum pressure developed, for to the surface load lifted it is necessary to add the force required to separate the clay cylinder, and also the frictional resistance to heaving which is not a factor when soils freeze in the ground. Moreover, an ice layer must begin to form at some favorable point and then spread outward rather than form across the entire cross section of the cylinder instantaneously. The clay used in these tests, as it comes from the mine, has a tensile strength of about 5 kilograms per square centimeter. Therefore, the maximum pressure was probably well over 14 kilograms per square centimeter, but it is doubtful if it appreciably exceeds 15 kilograms per square centimeter.

The amount of heave in open systems decreases with increase in pressure, and the maximum load which may be lifted increases with decrease in particle size. With finer material it should be possible to lift somewhat heavier loads, but with much decrease in size of particle the material would approach colloidal dimensions and become highly impermeable. Lack of permeability may be compensated for in part by slower freezing.

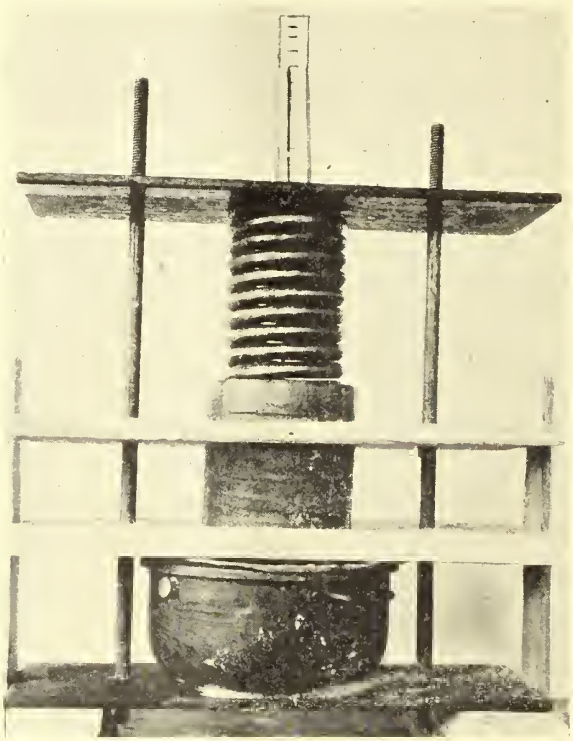


Figure 20.—Apparatus for Freezing Clay Cylinders Under Pressure

The limit to the load that may be lifted by frost heaving in an open system is not due to inability of ice crystals to grow under higher pressure, but to the failure of the water supply. This fact was demonstrated by supplying the water under greater pressure. The upper part of a strong container was filled with well-compacted moist clay and the lower part with water. (See fig. 22.) They were separated by a perforated disk of hard rubber, with filter paper resting on top, the disk being supported by a steel spring resting on the bottom of the container. The lower part of the container stood in a can of impervious cement, while the upper part was strengthened by surrounding it with wood strips held in place by steel bands.

The apparatus was placed in the refrigerator and buried to the top of the container in dry sand. A pressure of 14 kilograms per square centimeter was applied to the clay by means of a piston inserted in the top of the container, the pressure being measured by the compression of a calibrated spring. Because of the resistance of the smaller spring the water in the bottom of the container was under a pressure of less than 10 kilograms per square centimeter. As freezing progressed water was drawn into the clay to form ice layers and the springs were compressed until the final pressure on the clay was about 15 kilograms per square centimeter. The total thickness of the ice was between 2 and 3 centimeters (see fig. 23) whereas almost no heaving could be obtained when this clay was frozen under similar pressures but with the water under atmospheric pressure only.

THE MECHANICS OF FROST HEAVING IN OPEN SYSTEMS DISCUSSED

The mechanics of the process by which ice crystals, growing in open systems, are able to exert pressure and overcome resistance has been discussed in some detail in a recent paper [8] from which the following account is largely abstracted.

In indurated clay, or in clay that has been thoroughly consolidated artificially, the layers of segregated ice are clear (for the most part), solid, and very sharply separated from the clay. The ice usually shows a satiny luster because of the parallel orientation of the ice prisms and the presence of small filiform cavities filled with air, oriented in the direction of crystal growth. Both thickness and spacing of the ice layers tend to increase with distance from the cooling surface. The total thickness of the ice layers, as close as can be measured, equals the amount of surface uplift; and the water content of frozen clay between the ice layers is the same as that of clay below the depth of freezing. These observations prove that heaving in such tests is due to the formation of ice layers, the freezing of interstitial water being of little or no importance. In the test shown in Figure 14, A a sample taken just below the frozen material and three samples taken between successive ice layers gave 22, 22.1, 21.8, and 22 per cent water, respectively. Heaving was as uniform (see fig. 24) and not intermittent, which proves that a new ice layer began to form below as soon as the layer immediately above stopped growing.

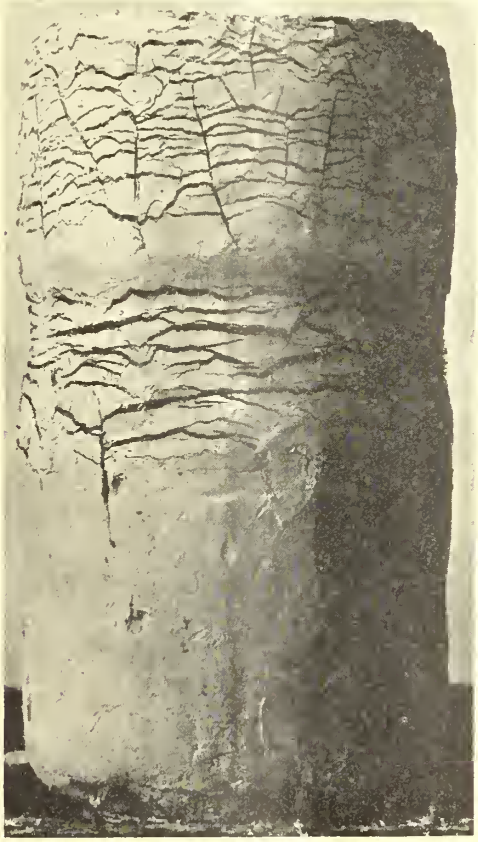


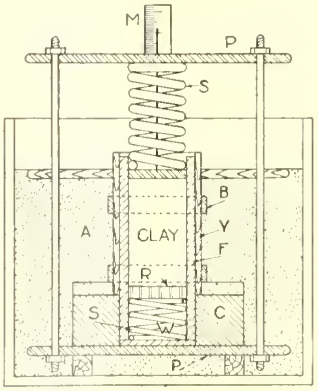
Figure 21.—Tightly Packed Clay Frozen Under a Load of 7.31 Kilograms Per Square Centimeter

In tests with unconsolidated clays the ice layers were not so sharply defined, and, in most cases, it was not possible to estimate accurately the thickness of the ice, but, here also, heaving must be due primarily to the formation of segregated ice.

"A growing ice crystal is in contact with a thin film of water similar to the adsorbed layer which forms on many other solids that are in contact with water. As a molecule in the film is oriented and attached to the crystal it is replaced by another from the adjacent liquid, thus maintaining the integrity of the film. If resistance to growth is offered by a solid body such as a soil particle, the pressure is exerted through the thin film of water that separates them. This film may consist of a single layer of molecules; but I am inclined to think that it is somewhat thicker, for the molecules in it possess considerable mobility. A unimolecular layer could not be expelled by pressure alone; thicker layers could be reduced through expulsion of some of the molecules under pressure, though this is probably resisted by the strong attractive forces. After the available water has been exhausted the film may be frozen, but it does not freeze readily.

"The orientation and attachment of a molecule to the crystal is accompanied by a slight repulsion, proportional to the change in volume; but this can not be considered an essential factor in the process, for liquids that freeze with decrease in volume give pressure effects in open systems similar to those obtained with water.

"Cohesion is greater between the molecules in the films and between these molecules and the ice than it is between water molecules that are not similarly located close to ice crystals. Since no outside force is competent to push molecules of water into the film between the growing ice crystal and the resisting solid, the crystal must be displaced relative to the adjacent solid because water is pulled in between them; and this is made possible by the high intermolecular attraction, which prevents the separation of the water molecules.



A—Dry Sand
B—Steel Bands
C—Cement
F—Fiber Container
M—Graduated Scale
P—Steel Plate
R—Hard Rubber Disk
S—Steel Spring
Y—Wood Strips
W—Water

Figure 22.—Apparatus for Freezing Clay Under Heavy Pressure with Reservoir of Water Under Less Pressure

"* * * Since ice crystals growing in an open system are able to overcome a resistance many times as great as atmospheric pressure, the water in such cases is actually pulled into the nourishing film under high tension.

"* * * A water column of large cross section is placed under tension with difficulty, for the water is superheated when under negative pressure, even at temperatures below 0° C., and the formation of a gas phase immediately breaks the column. Extremely slender columns or filaments and thin films are more

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[8] Stephen, Taber, "The Mechanics of Frost Heaving," Jour. of Geol., Vol. XXXVIII (1930), pp. 303-317.

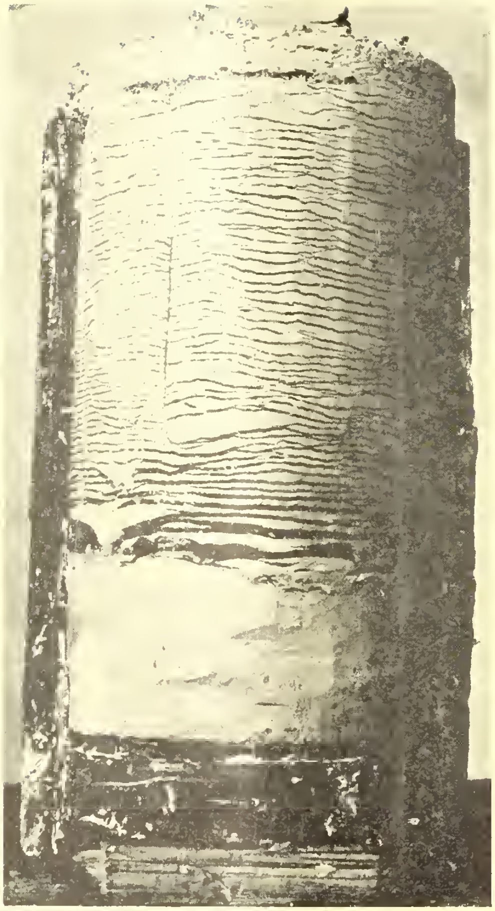

Figure 23.—Clay Frozen Under Pressure of 15 Kilograms Per Square Centimeter in the Apparatus Shown in Figure 22

easily maintained under tension than larger ones, and this is the form in which water is present in clays.

“* * * during the growth of an ice layer the voids in the adjacent underlying clay, beginning with the larger ones, tend to fill gradually with ice and thus increase the work that must be performed in supplying water to the ice layer. When resistance becomes too great the flow of water to this ice layer stops and a new layer of ice begins to form near the bottom of the zone of frost penetration. More of the water in the clay between the ice layers freezes as the temperature gradually becomes lower, some of it possibly migrating to the ice layers; * * *

“An ice prism in one of the layers continues to grow at the base where it is exerting pressure against an adjacent soil particle as long as water molecules can enter the separating film and be attached to the crystal. If the soil particle is very small, the molecules travel only a short distance through the film to reach the points where they are attached. If the soil particle is larger, it takes longer for the molecules to reach their points of attachment; and meanwhile freezing may extend downward around the particle so as gradually to inclose it in ice. As soon as crystal growth is checked at any point the temperature of the adjacent soil particle begins to fall, for heat is no longer liberated at this point by the conversion of water into ice; and since water is a poorer conductor of heat and has a higher specific heat than the minerals present in soils, the temperature of the bottom of the soil particle will reach the freezing point sooner than the water with which it is in contact. This helps to bring about the inclusion of the soil particle in the ice. The formation of layers of segregated ice is possible only when the soil particles are excluded by the growing crystals, a process which is aided by slow freezing, for then the molecules have more time in which to penetrate between crystal and soil particle.”

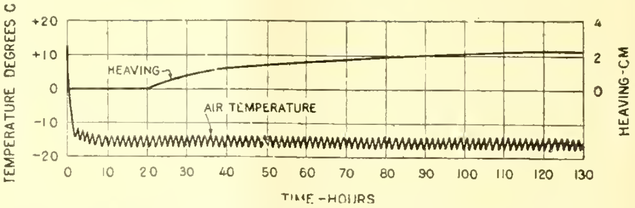

Figure 24.—Graphs Showing Air Temperature and Rate of Heaving During Freezing of the Clay Cylinder Shown in Figure 14, A

The growth of ice layers is stopped by lack of water supply, which may be due to rupturing of the upward moving filaments and films of water under tension or to the inability of molecules to enter films that are under pressure. Pressure tends to reduce the thickness of the nourishing film by expelling some of the molecules, and since the expulsive forces are increased the attractive forces must likewise be increased by lowering the temperature, else molecules can not enter the film. Pressure decreases molecular mobility in the film, retards crystal growth, and lowers the freezing point. The growth of ice crystals under pressure in open systems is possible because water occupying very small voids can be undercooled.

FREEZING OF SOIL A FACTOR IN THE DESTRUCTION OF PAVEMENTS

Uniform heaving of the subgrade results in little or no damage to pavements, but differential heaving may be very destructive, especially to thin, weak pavements and to those that are badly worn. The factors which determine frost heaving, such as texture and composition of soil, water supply, rate of freezing, and pressure, have all been discussed above. Local differences in any of these factors will tend to cause differential heaving; but differences in the soil and in the amount of water available are probably the commonest causes.

The importance of difference in soil texture is illustrated by the following experiments. A carton with perforated bottom was tightly packed with fine sand on one side and clay on the other, by inserting a partition down the middle and gradually withdrawing it as the soils were tamped into place. The carton stood in sand which was kept saturated with water throughout the experiment. After the soils had become moist on top the carton was buried to the top in dry sand, so that freezing took place from the top down. At first only the clay was appreciably heaved, but when the frozen crust became thicker, and therefore stronger, the sand was lifted along with the clay. As layers of segregated ice developed the underlying unfrozen clay was pushed down and under the sand, thus helping to force it up. The experiment was discontinued after the bottom of the carton had been forced out and to one side by the movement of the clay, as shown in Figure 25, A. If this experiment could have been made on a large enough scale no heaving of the sand would have occurred except in the immediate vicinity of its contact with the clay.

Another experiment was performed in the same way except that after the sand and clay had become saturated with water the carton was sealed at the bottom by standing it in a tray of molten paraffin. On freezing, layers of clear ice developed in the clay (see fig. 25, B), the water to form them being drawn out of the clay in the lower part of the carton and also from the sand. The withdrawal of water resulted in the formation of shrinkage cracks in the lower part of the clay, and the lower half of the sand was so nearly dry that it flowed readily. The large cavity shown in the photograph was caused by the dislodgment of dry sand, and more of the sand collapsed just after the picture was taken. The upper part of the sand was thoroughly cemented by ice which filled all voids. Because of differential heaving the frozen soil cylinder was gradually tilted away from the clay side, causing the wall of the carton to pull apart by tension. The inclination of the ice layers downward toward the sand may be in part due to this tilting, but it is largely to be explained by the fact that the freezing isotherm moved downward faster in the sand, where less water was being frozen than in the clay, so that some heat was conducted laterally from the clay into the sand.

These experiments show that, as a result of frost heaving, pavements are likely to be cracked and the sections differentially displaced at places where the subgrade soil changes from sand to clay, a change that occurs rather frequently in parts of the glaciated region. Also, where there are lenses of clay in sand, or vice versa, the excessive heaving over the clay may be caused by water drawn in part from the adjacent sand.

Since pressures of nearly 15 kilograms per square centimeter may be obtained through the freezing of pure clays in systems that are open with respect to water, a small mass of clay inclosed in nonheaving subgrade material may result in the uplift of a relatively large section of pavement, and thus subject it to stress sufficient to cause rupture. When sections of pavement are separated from the subgrade by differential heaving, air enters to form an insulating layer that retards the freezing of soils at these places as compared with those where the soil remains in contact with the pavement. As a result of differential heaving, pavements have sometimes been separated from the subgrade so that it was possible to see from one side of the highway to the other under the pavement. These facts emphasize the necessity of thoroughly mixing the subgrade material so as to make it uniform in texture and composition.

The amount of heaving that accompanies the freezing of soil is limited by the supply of water available, including the water which may be drawn from below the depth of freezing as well as that which was originally present. A uniformly dry subgrade does not heave and a uniformly wet one would result in uniform rather than differential heaving. Local differences in the moisture content of soils and especially differences in the position of the water table relative to the surface are important as causes of differential heaving. This condition may frequently be remedied by drainage designed to lower the water table under subgrades where it comes too close to the surface, but deep drainage is necessary in soils where the capillary fringe rises high above the water table.

Local differences in the rate of cooling due to differences in soil cover are a minor cause of differential frost heaving. Ordinarily the soil forming the shoulders of a road, if wet, will begin to freeze before the soil lying under the pavement; but when the ground is dry, or is covered with snow while the pavement is cleared for traffic, the reverse would be true. In suitable soils the water will be drawn toward those points where freezing is going on and away from those areas where freezing has not begun.

When the road shoulders begin to heave before soil freezes under the pavement, the latter may possibly be lifted because of cohesion between frozen soil and pavement. This would result in the introduction of an insulating layer of air between pavement and subgrade, which would further retard freezing under the pavement.

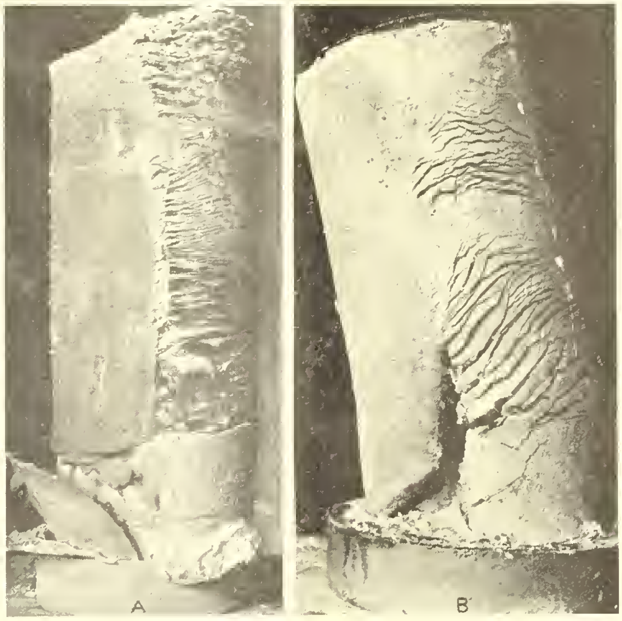

Figure 25.—Experiments Showing Differential Effects of Freezing in Sand and in Clay. A.—Frozen Cylinder Half Sand and Half Clay. Much Segregated Ice in Clay but not in Sand. B.—Differential Displacement of Cylinder, Due to Segregation of Ice in Clay but not in Sand. Cavity Caused by Dislodgment of Dry Sand

The process is easily illustrated experimentally. A smooth rectangular block of iron was embedded in clay so as to be flush with the surface, the clay being packed in a large carton which stood in wet sand. When heaving began the iron block was not, at first, displaced with reference to the surface, for it was lifted from its bed with the formation of an air pocket below. Two days later, because of slow vaporization of the ice, cohesion between the smooth metal and the frozen clay was reduced, and the block dropped into the pocket so that it was 1.3 centimeters below the surface of the clay.

From the meager observations that the writer has been able to make, it seems likely that heavy cement pavements are seldom if ever lifted because of heaving of soil forming the shoulders; but this is a question requiring further field investigation. The differential heaving of subgrade and shoulders does, however, help to explain the breaking up of asphalt macadam and surface treated roads along the edges.

After a road surface becomes badly worn and pitted, heaving of the subgrade will begin at those points where the pavement is thinnest, and gradually lift it until an insulating air layer surrounds the area of heaving. As heaving continues the stresses tending to cause fracture increase until sooner or later weak pavements collapse around the point of uplift. This is probably the explanation of most of the so-called frost boils that form on asphalt macadam and other relatively weak types of road surfacing, but frost boils may also be caused by differential heaving due to local differences in the texture and composition of the subgrade soil.

THAWING OF SOIL ALSO A FACTOR

Experiments prove that freezing tends to concentrate water near the surface even when the soil contains relatively little water. Under favorable conditions excessive quantities of water may be drawn upward and concentrated near the surface in the form of segregated ice. More water can be introduced into a soil that is in place in this way than by any other natural process. When thawing occurs this excessive amount of water may be disposed of in two different ways.

1. If the air temperature remains barely below freezing for a long enough time, a deeply frozen soil will thaw gradually from the bottom, because of the outward conduction of heat from the interior of the earth, and the water may then return to the underground reservoir from which it was drawn. But since water is pulled toward the surface during the freezing process by a force that is greatly in excess of the forces causing its downward movement in soil, the water may not be removed as rapidly as melting takes place unless the latter is very slow.

2. On the other hand, if the air temperature is well above 0° C. so that melting occurs from the surface, the water set free can not escape downward through soil voids until thawing is practically complete. Under these conditions the water has to remain in the soil until it can drain out to the surface and flow away, which is a rather slow process.

The thawing of deeply frozen soil, of course, usually takes place from both top and bottom, but sometimes one type of thawing predominates and sometimes the other. When a soil containing much segregated ice thaws rapidly from the top because of a sudden rise in air temperature, the soil becomes supersaturated with water.

A heavy snow fall before the ground freezes forms a protective blanket, and if the ground remains well covered throughout the winter relatively little soil freezing occurs. Even though the snow comes after the ground has frozen to a considerable depth, it may prevent rapid removal of heat, and therefore enable the soil to thaw slowly at the bottom of the zone of frost penetration. If the snow melts rapidly in the spring much of the water runs off on the surface and comparatively little enters impervious clay soils. The soil may become saturated close to the surface, but it does not become supersaturated, as may happen when segregated ice melts. This explains why common dirt roads in areas of clayey soil are usually in worse condition after a spring thaw that follows an open winter than after a winter when the ground has been heavily blanketed with snow.

It is difficult for water from rain or melting snow to accumulate immediately under impervious pavements in quantity sufficient to saturate the soil, especially where reasonably good drainage has been provided; but as a result of freezing where the conditions are favorable, water sufficient to supersaturate the soil may be concentrated under the pavement. Keeping pavements cleared of snow while the ground on each side is buried should increase the amount of segregated ice formed under the pavements, for water may then be drawn in part from the soil at the sides.

Since the mobility of soils under stress is increased by the presence of excessive amounts of water, superficial landslides occasionally occur during spring thaws. Clay containing much water will creep or flow out from under a heavy weight, such as a pavement, causing it to slowly settle. Where drainage facilities are good, so that water from a melting ice layer escapes rapidly, soil particles may be entrained and carried out with it.

Slow removal of material from under a pavement by soil creep or by entrainment of particles in draining water results in unequal settling, but cracks due to this cause may develop only under heavy traffic, perhaps long after the freezing and thawing occurred. Some cracks are probably due to the cumulative effects of several seasons of freezing and thawing. Unequal settlement after thawing has not been appreciated as a cause of the failure of pavements.

Rapid thawing from the surface downward, in soils where segregated ice forms, tends to accentuate all of the engineering troubles ordinarily associated with poor drainage, but methods of draining that satisfactorily protect road pavements in warm regions may be inadequate on some soils in regions of deep frost penetration.

THAWING AND REFREEZING DESTRUCTIVE TO ROAD PAVEMENTS

The formation of segregated ice in soils, which often results in differential heaving, is favored by a high water content, for water is then immediately available for the growth of ice crystals. Subgrade soils, protected by impervious pavements, are not easily saturated by rains or by water from melting snow, if ordinary drainage has been provided; but in suitable soils under favorable conditions water may be pulled up from below and concentrated under a pavement in the form of ice, which, on melting, may set free sufficient water to saturate or even supersaturate the soil. Therefore, a sudden drop in temperature after a spring thaw has left an excessive amount of water in the soil is apt to be more destructive to pavements than the slow freezing of an entire winter.

The effects of refreezing after a thaw are also accentuated by the fact that the first freeze leaves the soil in a more or less loosened or expanded condition. Experiments have demonstrated that heaving is greater on unconsolidated clays than on those that are thoroughly consolidated. Bituminous macadam and other relatively weak types of surfacing material seem to be especially susceptible to differential heaving brought about by a sudden freeze after a spring thaw.

DISTRIBUTION OF DAMAGE BY FROST HEAVING ERRATIC

Engineers have found it difficult to explain the apparently erratic distribution of damage from frost heaving. A pavement may be badly damaged the first winter after it is laid and then suffer relatively little injury from frost heaving during a series of succeeding winters. Other pavements are sometimes seriously damaged after having passed through several winters with little or no trouble. In one locality a certain type of soil may give much trouble while in another locality roads constructed on the same type of soil are said to be virtually immune to frost action. These apparent inconsistencies arise largely from the fact that damage by frost heaving is controlled by not one but several variables.

A new pavement may sometimes suffer greater damage from frost heaving because the subgrade soil has not become thoroughly consolidated. Repeated tests have shown that the rate of heaving and the ratio of heaving to depth of freezing are much greater on loosely consolidated soil, than on the same material when thoroughly consolidated. (Compare the tests shown in figs. 14, A and 14, B.) Some pavements suffer little damage until after they have become weakened or worn and pitted from service.

With the same soil and locality the heaving varies greatly during different winters because of differences in the position of the local water table, differences in precipitation, and differences in temperature. And in considering temperature differences, the length of a cold spell and the minimum temperatures reached may often be less important than the fluctuations in temperature that bring about freezing and thawing and result in the concentration of water near the surface.

Some winters little soil freezing occurs because of a protective blanket of snow, while during open winters frost penetration is relatively deep.

In a flat region where the water table practically parallels the surface and there is no appreciable variation in the texture and composition of the soil over large areas, such heaving as occurs is likely to be uniform, and therefore to cause little damage to pavements. In another area, having the same type of soil but more relief, the depth of the water table will vary; and this, as well as other factors, causes differences in the available water supply, and consequently differential heaving.

OBJECTS BURIED IN SOIL SOMETIMES DISPLACED BY FREEZING AND THAWING

When soils, because of freezing and thawing, become supersaturated with water, buried objects, especially if smooth and of high specific gravity, tend to sink. This downward displacement at the time of a single thaw is usually small, but repeated freezing and thawing bring about appreciable effects. This probably explains the gradual settlement of gravel from under pavements that has been observed in certain districts.

During the heaving of a soil, cavities tend to form immediately over buried objects because less segregated ice forms there than at points where impermeable material does not separate growing ice crystals from the water supply.

When a large object is gripped in frozen soil and lifted from its bed by heaving, a cavity develops beneath. Soil may accumulate in the cavity while the object is suspended, especially if thawing takes place chiefly from the bottom of the frozen layer, so that the object can not settle back to its former position. This is probably the main factor in causing certain objects, such as culverts, gradually to work upward toward the surface.

METHODS OF DETERMINING THE RELATIVE HEAVING PROPERTIES OF SOILS DESCRIBED

In order that suitable precautions may be taken it is often desirable to ascertain how a soil will behave when subjected to freezing, and in this connection the most important fact to be determined is the amount of heaving that can occur when freezing conditions are most favorable. It has been shown that the texture and composition of a soil, and especially the percentage of clay present, are the main factors. When known they give a pretty good indication of how the soil will behave. A larger number of freezing tests, accompanied by complete physical analyses on a greater variety of soils, should make it possible to forecast with some accuracy the behavior of a soil on freezing.

In the present investigation two methods have been devised for measuring the relative heaving qualities of soils. The first method used was to determine the ratio of surface uplift to depth of freezing. It may be used whenever it is possible to measure accurately the amount of heaving. The main objection to the method is the difficulty of determining the exact depth of frost penetration in clays, and in laboratory tests, which are limited in duration to a few days, this difficulty may introduce a relatively large error. Also, it is not, as a rule, possible to remove a test from the refrigerator as soon as heaving stops.

The second method is based on the rate of heaving. Most soils, when frozen at constant air temperature in the apparatus described above (see fig. 3), heave uniformly for several hours after freezing begins, so that the graphs drawn by the recording pen are nearly straight lines. These graphs would have shown more curvature if it were not for the fact that the containers were surrounded by dry sand which did not heave.

Where either method is used, strictly comparable results can be obtained only when every precaution is taken to perform the tests under exactly the same conditions of cooling, of water supply, and of resistance to heaving, which includes friction between soil and container as well as surface load. To insure similar conditions of cooling, the soil containers should be the same height above the floor of the refrigerator box so that there will be the same amount of insulating material under them; and the initial temperature of the soil, the minimum temperature, and the rate at which the surface temperature drops should all be the same. Differences of only a few degrees in the room temperature produce no appreciable effects.

Frictional resistance between soil and container varies considerably with different soils, and as yet no satisfactory method of equalizing this factor has been found. When soils freeze in the ground frictional resistance is seldom a factor. In a few tests there was a tendency for the soil to push out the bottom of the container and displace the underlying sand, but a method has been devised which will obviate this difficulty in future tests.

South Carolina Cretaceous white clay was used in most of the preliminary freezing tests because it is a very pure kaolin, uniform in texture, and practically free from grit. Tests with this clay also photograph well. As compared with other soils that have been tested, its rate of heaving is exceptionally high, although the clay from St. Peter, Minn. (see p. 116), closely approaches it. The latter heaves at the rate of 0.76 millimeter per hour while the South Carolina clay under the same conditions heaves 0.83 millimeter per hour.

MEASURES DISCUSSED FOR PREVENTION OF DAMAGE CAUSED BY FREEZING AND THAWING OF SUBGRADE SOILS

No panacea can be prescribed for all of the ills associated with the freezing and thawing of soils. Rather, each locality where frost heaving is troublesome should be treated as an individual problem, for a satisfactory solution in one place may prove impractical in another. It is now possible to determine by laboratory tests the relative heaving properties of soils, but in each case a field investigation is necessary to determine the other factors involved in frost heaving, such as the possibility of freezing taking place as in a closed system and the probability of water being available for excessive heaving.

Some places which are subject to excessive differential heaving can be avoided in locating the highway, but in most cases this would be impractical. The use of smaller and stronger sections reduces cracking but does not prevent spalling and unevenness at the joints due to differential displacement. Better drainage is often beneficial, though drainage alone will not remove water occupying extremely small capillary voids in soil, or prevent the drawing up of more water during the growth of ice crystals.

Sodium silicate mixed into clay lowers the freezing point, increases the percentage of water that does not freeze, and reduces permeability. In practical use it would be slowly leached out of soils that are alkaline. In acid soils sodium silicate decomposes with the precipitation of insoluble gelatinous silica, which on drying would act as a cement and help to bind the soil particles together. The presence of carbon dioxide in the ground water would be sufficient to bring about the precipitation of the silica.

The troubles resulting from the formation of segregated ice under pavements can be entirely prevented if, in addition to the usual methods of draining, a thick layer of coarse material is introduced under the pavement extending down to the extreme depth of frost penetration. The only objection to this method is the cost, but in those localities where heaving is bad and the expenditure is warranted it should be used.

Addition of sand to the subgrade material will prevent ice segregation, and, fortunately, many soils naturally contain sufficient sand. On soils that do not heave readily the weight of the pavement becomes a factor in preventing uplift. The admixture of colloidal material, such as bentonite, reduces heaving by decreasing permeability, but the increase of colloids in a soil tends to cause greater swelling and shrinking when water is absorbed or lost by evaporation. Uniformity of texture in subgrade soils is essential if differential heaving is to be avoided, and this end can usually be obtained by care in mixing when additional material has to be introduced.

In the future solution of problems connected with the freezing and thawing of soils the most promising opportunities for research at the present time lie in field investigations. Accurate observations are needed, under widely varying conditions in different parts of the country, of the topography, the distribution and occurrence of ground water, and other pertinent factors at points where heaving is excessive and also at points where little or no heaving occurs on soils favorable for ice segregation. The investigations can best be made by geologists who are familiar with the occurrence and movement of ground water in different types of soil and who have studied the processes involved in frost heaving.

ENGINEERS FROM ALL PARTS OF WORLD TO PARTICIPATE IN INTERNATIONAL ROAD CONGRESS

More than 50 nations have accepted the invitation of the United States Government to participate in the Sixth International Road Congress to be held in Washington, October 6 to 11, and officials in charge are entering the final stage of their preparations to receive the large number of delegates expected.

The congress will be held under the auspices of the Permanent International Association of Road Congresses, with headquarters in Paris, and under the immediate direction of the American Organizing Commission, Roy D. Chapin, president, with offices in Washington.

Seventy-six separate reports, printed in French, German, Spanish, and English, the four official languages of the congress, will form the basis for discussion at the sessions in October. The reports represent the studied opinions of 170 of the principal world authorities on the moot questions of highway construction, design, material, maintenance, administration, finance, and traffic.

More than a year ago the several topics to be studied at the congress were decided upon and the chief exponents of the various questions assigned to write upon them. The result has been a series of articles and papers that represent most that is known to-day of the twin sciences of highway cosntruction and automotive transportation.

For months a corps of translators has been engaged in the task of preparing the reports to the congress in four different languages. The English and Spanish translations have been made in Washington under the supervision of engineers and editors of the United States Bureau of Public Roads, while the French and German versions have been made in Paris under the immediate direction of M. Le Gavrian, secretary-general of the association.

With the task of translation and printing completed, the reports are being mailed to the far corners of the earth, so that delegates coming to Washington this autumn may fortify themselves to support or to attack the theories and conclusions set forth in the reports. Each member of the association will receive a collection of the reports in that one of the four languages with which he is most familiar. Membership in the association is essential to participation in the congress.

The net result of this distribution will be an intensive, simultaneous world study of highway problems preparatory to the serious discussions of the congress itself. Following the congress, the proceedings, containing the debates, will likewise be published in the four languages.

Seven general reporters have consolidated the individual papers, giving extensive résumés of the discussions on the six major questions on the agenda.

AMERICAN ENGINEERS URGED TO ATTEND CONGRESS

Highway engineers and officials and representatives of organizations and industries interested in highway construction and transporation are urged to attend the congress. A temporary membership may be obtained through the American Organizing Commission, 1723 N Street NW., for $5. The volume of reports and general conclusions is now being distributed to all classes of members and these reports are available only to members of the congress.

Details concerning membership in the congress, the agenda of the congress, and the program which has been adopted will be found in the two preceding issues of Public Roads.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.
220D. Road Models.
257D. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
1216D. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 94C. T. N. T. as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
*93M. Direct Production Costs of Broken Stone. 25c.
*109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails. 10c.

SEPARATE REPRINTS FROM THE YEARBOOK

No. 914Y. Highways and Highway Transportation.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Connecticut.
Report of a Survey of Transportation on the State Highway System of Ohio.
Report of a Survey of Transportation on the State Highways of Vermont.
Report of a Survey of Transportation on the State Highways of New Hampshire.
Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio.
Report of a Survey of Transportation on the State Highways of Pennsylvania.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

JULY 31, 1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
		$	$				$	$				$	
Alabama	2,185.0	1,481,008.51	727,341.58	68.0	8.2	74.2	13,545.40	5,772.70				4,138,887.66	Alabama
Arizona	81[illegible].4	3,941,758.03	2,958,897.65	128.2	148.5	277.7	88,818.86	88,670.19	4.0	3.6	7.6	1,891,906.00	Arizona
Arkansas	1,744.0	5,685,239.28	2,630,989.23	170.6	42.8	213.4	1,695,042.02	840,648.39	62.0	12.8	74.8	1,389,142.48	Arkansas
California	1,884.3	7,084,139.11	2,832,920.88	122.8	27.8	150.6	2,836,368.85	1,312,096.03	70.9	2.2	73.1	1,068,822.41	California
Colorado	1,183.2	5,130,165.48	2,695,453.16	184.1	69.4	253.5	670,833.60	418,069.89	24.3	27.8	52.1	2,238,694.88	Colorado
Connecticut	243.3	2,595,633.60	1,099,282.57	14.0		14.0	670,974.87	156,815.00	10.5		10.5	623,875.34	Connecticut
Delaware	261.7	1,030,918.80	496,933.02	50.3		50.3	135,092.62	87,546.31	7.0		7.0	83,675.18	Delaware
Florida	503.5	5,127,463.81	2,358,944.87	97.1	5.5	102.6	501,391.49	250,595.73	4.7		4.7	1,434,128.83	Florida
Georgia	2,671.3	5,047,427.55	2,395,378.57	152.0	81.2	233.2	1,718,814.31	812,471.03	41.0	44.1	85.1	2,412,314.33	Georgia
Idaho	1,201.2	1,410,568.96	848,598.85	86.3	26.1	112.4	1,582,028.78	622,000.00	97.4	40.4	137.8	682,860.89	Idaho
Illinois	2,048.2	16,060,870.53	8,521,844.65	541.0	24.5	565.5	4,605,379.95	2,185,005.77	127.3	38.2	165.5	3,560,793.38	Illinois
Indiana	1,493.8	4,841,069.61	2,325,038.73	150.9		150.9	903,168.23	375,647.33	26.2		26.2	2,098,918.42	Indiana
Iowa	2,968.4	7,461,183.82	3,175,262.01	66.0	190.2	256.2	808,633.66	335,067.84	11.9	15.1	27.0	5,310.32	Iowa
Kansas	2,644.6	5,482,334.35	2,516,606.81	256.9	27.8	284.7	852,946.78	435,946.80	33.4	90.4	123.8	1,934,282.56	Kansas
Kentucky	1,521.6	3,850,648.93	1,895,449.74	135.6	5.5	141.1	4,852,314.89	2,213,273.11	45.8	228.0	273.8	115,709.00	Kentucky
Louisiana	1,349.4	5,068,523.05	2,466,932.87	153.8	17.4	171.2	1,583,046.39	773,792.08	49.0	8.3	57.3	1,222,014.68	Louisiana
Maine	535.1	3,012,394.14	1,089,092.97	71.7	1.5	73.2	550,143.65	225,444.75	18.8		18.8	1,206,422.28	Maine
Maryland	630.7	2,050,152.03	989,662.14	52.4	12.6	65.0	546,938.32	288,773.00	8.4	5.8	14.2	5,643.03	Maryland
Massachusetts	684.0	5,641,642.10	1,743,365.72	70.7	2.6	73.3	631,733.63	157,605.00	10.5		10.5	1,788,720.32	Massachusetts
Michigan	1,624.8	9,322,319.60	3,942,587.95	212.1	30.5	242.6	521,780.27	242,475.00	25.7		25.7	2,789,104.43	Michigan
Minnesota	3,818.8	11,498,661.62	4,092,278.31	240.5	255.0	495.5	625,592.49	218,575.40	11.1	14.8	25.9	46,352.03	Minnesota
Mississippi	1,820.7	1,795,493.20	691,571.41	55.0	7.7	62.7	40,111.51	20,055.75	.1		.1	3,532,317.45	Mississippi
Missouri	2,500.8	8,340,247.74	2,815,535.03	128.8	56.5	185.3	2,640,363.32	909,414.31	38.8	27.7	87.8	701,743.95	Missouri
Montana	1,735.3	7,725,086.44	4,358,263.83	535.8	53.5	589.3	1,460,794.59	807,560.17	137.2	30.8	168.0	3,004,098.65	Montana
Nebraska	3,718.0	8,828,415.54	3,138,559.32	243.4	129.6	373.0	1,270,697.18	510,753.29	51.7	83.0	134.7	2,160,257.26	Nebraska
Nevada	1,179.3	949,063.54	840,337.62		162.0	182.0	186,710.29	185,628.15		35.1	35.1	723,022.82	Nevada
New Hampshire	352.7	1,586,106.43	573,594.89	37.3		37.3	348,913.98	108,005.00	5.4	1.6	7.0	83,787.56	New Hampshire
New Jersey	518.3	5,535,034.60	1,348,431.32	58.2		58.2						677,248.78	New Jersey
New Mexico	1,823.3	3,701,393.91	2,521,487.98	191.5	50.7	242.2	741,524.24	525,087.05	26.4	8.2	34.6	605,148.35	New Mexico
New York	2,461.0	23,966,187.78	4,780,455.00	319.1		319.1	9,577,950.00	1,743,722.50	115.6		118.6	5,542,097.86	New York
North Carolina	1,782.3	4,185,218.02	2,070,849.62	158.9	28.9	187.8	1,287,723.62	522,158.25	52.5	1.9	54.4	1,902,915.11	North Carolina
North Dakota	4,250.6	2,284,275.60	1,203,167.63	354.7	157.8	512.5	1,350,377.84	674,894.93	158.9	204.2	363.1	1,180,507.18	North Dakota
Ohio	2,215.5	20,751,651.68	6,877,958.30	387.5	29.6	417.1	5,465,862.46	1,861,314.30	69.9	18.0	87.9	91,458.09	Ohio
Oklahoma	1,895.8	4,792,879.33	2,200,512.88	146.6	61.5	208.1	2,015,660.15	943,834.11	51.4	38.2	90.6	131,500.73	Oklahoma
Oregon	1,138.5	6,011,274.27	3,518,306.10	226.0	97.3	323.3	811,874.55	366,669.70	55.6		55.6	25,209.58	Oregon
Pennsylvania	2,354.1	20,109,092.60	5,482,571.63	259.7	14.1	273.8	4,595,624.96	1,284,497.50	53.2		53.2	684,027.89	Pennsylvania
Rhode Island	195.2	1,505,655.94	586,064.33	21.7		21.7	520,359.91	148,125.00	5.8		5.8	333,199.35	Rhode Island
South Carolina	1,868.5	4,726,336.40	2,017,574.94	104.2	80.4	184.6	1,232,521.05	439,064.20	21.3	26.2	47.5	89,218.24	South Carolina
South Dakota	3,551.1	3,838,732.11	2,036,792.10	370.6	123.6	494.2	680,327.51	409,524.28	49.8	81.4	131.2	1,004,479.15	South Dakota
Tennessee	1,244.8	4,185,340.75	1,753,182.71	145.1	40.0	185.1	2,240,583.35	1,071,005.35	88.8	10.7	89.5	1,206,111.81	Tennessee
Texas	6,844.2	12,850,259.56	5,207,080.58	396.7	138.8	535.5	3,041,145.30	1,227,904.32	63.3	92.5	155.8	4,175,175.63	Texas
Utah	968.2	1,275,141.24	687,151.84	78.7	32.2	110.9	355,570.00	259,783.40	16.1	57.5	73.6	708,518.01	Utah
Vermont	255.6	2,153,795.06	778,211.61	42.4	2.6	45.0	315,892.82	45,829.37	5.7		5.7		Vermont
Virginia	1,472.5	4,607,646.48	2,258,205.29	216.8	13.6	230.4	538,272.40	248,143.54	20.2	3.0	23.2	573,299.83	Virginia
Washington	903.2	4,040,260.58	1,751,900.00	83.8	38.3	129.6	162,586.68	83,600.00	1.7		1.7	1,199,863.86	Washington
West Virginia	700.2	3,744,304.82	1,380,940.60	80.6	37.8	118.4	1,373,872.95	480,102.90	29.5	7.1	36.6	391,874.30	West Virginia
Wisconsin	2,256.8	8,102,049.75	3,241,883.29	187.1	48.2	245.3	1,218,486.12	555,875.00	21.4	15.8	36.3	480,173.33	Wisconsin
Wyoming	1,702.8	2,084,300.10	1,357,531.50	139.3	107.8	247.1	682,129.55	480,705.72	37.0	57.4	94.4	413,707.39	Wyoming
Hawaii	41.2	853,665.90	359,459.43	21.5		21.5	225,820.18	113,414.57	8.2		8.2	1,568,624.56	Hawaii
TOTALS	84,173.7	289,465,194.65	117,844,624.42	8,054.9	2,487.6	10,542.5	71,079,362.63	28,441,064.81	1,878.9	1,334.8	3,213.7	65,256,147.78	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE: 1930

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 7 ▽ SEPTEMBER, 1930

RUNWAY WITH ARTIFICIAL OBSTRUCTIONS USED IN IMPACT TESTS AT ARLINGTON

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 11, NO. 7 SEPTEMBER, 1930 G. P. St. CLAIR, Editor

TABLE OF CONTENTS

Page

Motor Truck Impact as Affected by Rubber Tread Thickness of Tires 133

The Interrelated Effects of Load, Speed, Tires, and Road Roughness on Motor Truck Impact . 139

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MOTOR TRUCK IMPACT AS AFFECTED BY RUBBER TREAD THICKNESS OF TIRES

REPORT OF COOPERATIVE TESTS BY THE BUREAU OF PUBLIC ROADS, THE SOCIETY OF AUTOMOTIVE ENGINEERS, AND THE RUBBER MANUFACTURERS ASSOCIATION

Reported by **JAMES A. BUCHANAN**, Associate Engineer of Tests, Division of Tests, U. S. Bureau of Public Roads [1]

> Since the tests reported in this article were conducted, a thorough investigation of the accuracy of the instruments used has been made by the United States Bureau of Standards, in cooperation with the United States Bureau of Public Roads. A summary account of this work was published in PUBLIC ROADS, July, 1930. The committee which guided the investigation recommended that the following statement regarding the accuracy of the data accompany this report:
>
> **The results obtained in the motor-truck impact tests are, in general, systematically from 10 to 15 per cent too low. A conservative estimate of the dispersion of the measured values of acceleration indicates that 90 per cent of the significant accelerations lie within less than 15 per cent of the mean, for tests involving artificial obstructions.**

THE experimental motor-truck impact tests which were reported in part in the June, 1926, issue of PUBLIC ROADS [2] have been continued and the scheduled tests on cushion and solid tires cut to various heights of tread rubber are now completed.

The Bureau of Public Roads, the Rubber Manufacturers Association, and the Society of Automotive Engineers have continued to cooperate in this work and the administration and test procedure in these tests are the same as in those previously reported. For specific and detailed information concerning instruments used and methods of testing, reference should be made to the earlier report.

The object of this particular phase of the impact investigations was to determine the effect of reducing the thickness of the tread rubber on the cushioning properties of typical solid and cushion tires. Briefly, the procedure followed was to equip the test truck with new tires and, under carefully controlled conditions of load and speed, to drive the truck over a test road artificially roughened with steel obstructions. For these conditions the vertical impact reactions were determined. The tires were then removed from the truck, placed in the tire cutting machine shown in Figure 1 (described later), and a definite depth of the tread thickness removed. This insured tires identical in all respects except that of the factor being studied (tread rubber thickness). These cut-down tires were then replaced on the test truck, the runs over the artifically roughened test road repeated and the impact reactions determined for this new tire condition. By continuing this program of cutting down and testing as far as was practicable with the various types of tire, data were obtained which show the relation between the loss of tread rubber and the resulting increase in impact reaction for each of the typical tires included in these tests.

APPLICATION OF RESULTS LIMITED BY CERTAIN FACTORS

There are certain limitations to these tests and it is important that they be clearly understood in order to apply properly the results obtained.

The tires of reduced tread thickness were cut down from practically new stock and the cutting process, by its very nature, made them true and round. Tires with tread thickness reduced by wear in service are likely to be worn to varying degrees along the circumference and probably have suffered more or less aging and hardening at the same time. Both of these differences would tend to make the cushioning properties of service-worn tires somewhat less than those of similar tires cut down to the same height by the process used in these tests.

The results presented in this report are based on total vertical impact reactions at the instant of impact. They do not take into consideration the horizontal or tangential component of the reaction at the road surface. From the standpoint of the pavement structure it is probable that it is the large vertical component of this force which is important.

The total vertical reactions are shown without any attempt to determine the pressure intensity or the distribution of pressure over the area of contact. The general trend of the data obtained in the impact tests indicates that, the load and other variables being equal, wide tires cause heavier impact reactions than narrow tires. It is recognized that, under given loads, different tires may have different areas of contact. Thus the total vertical reaction values are given as determined, without prejudice to those tires which through larger areas of contact may possibly tend to develop somewhat lower unit pressures. The effect of variations in unit pressures or pressure distribution over the area of contact on the stresses developed in a pavement is as yet a very uncertain matter.

Tire manufacture is in a state of constant development and improvement. Since the tires used in these tests were obtained the tendency has been to increase the height of tread rubber in both solid and cushion types. Therefore the tires included in this report are in a sense obsolete in that the most recent tires of these types have generally a somewhat greater thickness of tread rubber. Tire manufacturers state that the production of regular or low profile solid tires is decreasing, such types being superseded by high profile solid tires. However, the practical application of the data herein presented is not precluded by such changes if the actual dimensions are taken into account whe considering the influence of the tread-rubber thickness of any tire on the total vertical impact reaction.

---

[1] The bureau acknowledges, with appreciation, the valuable assistance of J. W. Reid, formerly with the Rubber Association, in planning the tests and securing the data.

[2] See Motor-truck Impact as Affected by Tires, Other Truck Factors and Road Roughness, Public Roads, vol. 7, No. 4, June, 1926, and also General Results of the Cooperative Motor-truck Impact Tests, Journal of the Society of Automotive Engineers, vol. 18, No. 6, June, 1926.

It is felt that this report will be most useful if it is considered as indicating only the general trend of the relations studied.

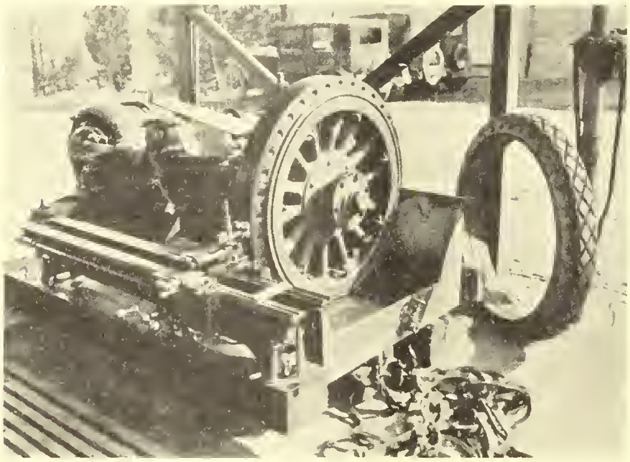

Figure 1.—Apparatus for Cutting Tires

TERMS DEFINED

To aid in a clear understanding of the matter under discussion, certain terms which appear frequently are defined as follows, reference being made to Figure 2:

Visible rubber or visible height of rubber.—The radial distance from the top of the steel flange of the rim to the extreme outer surface of the rubber, the measurement being taken at no load. In this report, visible rubber has been used to define tire height, it being the most readily determinable measure.

Over-all height or over-all sectional height.—The radial distance from the inside of the steel rim to the extreme outer surface of the rubber, the measurement being taken at no load.

Flange height.—The radial height of the steel flange of the rim, which is equal to the difference between the over-all height and the visible rubber. This flange height may be generally taken as seven-eighths inch without serious error.

Static load.—The total pressure exerted on a level surface by the tire of one rear wheel of a stationary motor vehicle.

Average impact reaction.—The impact force per rear wheel, which is the average for a series of certain representative test conditions of load, speed and obstruction (pavement roughness). It may be expressed as a pressure in pound units or as a multiple or percentage of the static wheel load.

DETAILS OF TESTS DESCRIBED

Tires.—The impact test data herein presented relate to three types of 36 by 5-inch dual cushion tire and two types of 36 by 4-inch dual solid tire equipment on the 2-ton test truck, and one type of 36 by 7 inch dual cushion tire and one type of 36 by 5 inch dual solid tire equipment on the 3-ton test truck. The tire equipments applied to the two trucks conformed with accepted practice at the time of the tests.

The cushion tires were all of the nonskid-tread designs and represented the high and low annular internal cavity and the radial external cavity types. The solid tires had smooth treads and represented the high-profile and the regular or low-profile types.

As a tire was cut down the depth of nonskid depressions was decreased or entirely eliminated and the face width of the tire increased with the depth of the cuts. In each case, the new tires were the only ones which had rounded profiles as all cuts were made from side to side parallel with the axis.

Speeds, loads, and obstructions.—The trucks were operated at various speeds from a minimum of about 3 miles per hour up to a maximum of about 20 miles per hour. The speeds varied in increments of about 3 miles per hour and it was possible to plot speed versus impact force and obtain curves from which the force at any given speed could be obtained.

Loads were applied in accordance with the capacity of the tires on the rear wheels, except in cases where trucks were empty. The capacity loads of the various tires as used in these tests were as follows:

All 36 by 5 inch cushion, 1,700 pounds per tire.
All 36 by 7 inch cushion, 3,500 pounds per tire.
All 36 by 4 inch solid, 2,000 pounds per tire.
All 36 by 5 inch solid, 3,000 pounds per tire.

The artificially roughened concrete test road at the Arlington experimental station was used to conduct these tests. On this road, which is straight and practically level, and has but slight crown, eight steel obstructions were bolted at about 30-foot intervals along a painted guide line. The obstructions listed below represent in a controlled, artificial way many of the surface roughness conditions presented by actual highways.

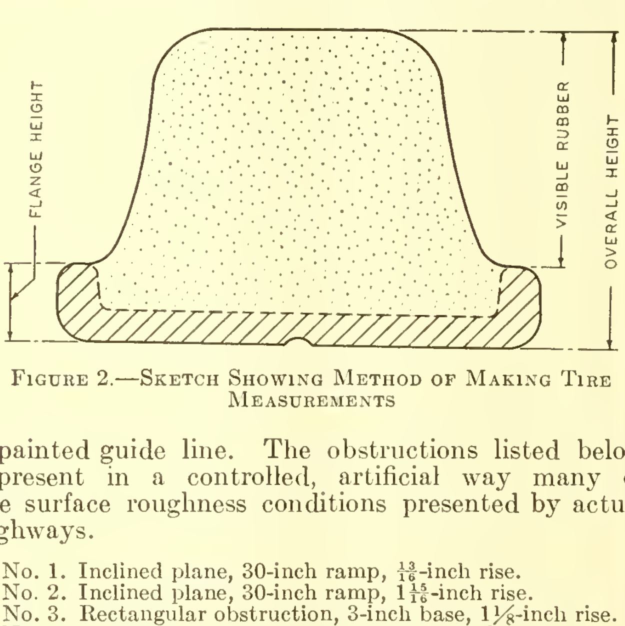

Figure 2.—Sketch Showing Method of Making Tire Measurements

No. 1. Inclined plane, 30-inch ramp, $\frac{13}{16}$-inch rise.
No. 2. Inclined plane, 30-inch ramp, $1\frac{15}{16}$-inch rise.
No. 3. Rectangular obstruction, 3-inch base, $1\frac{1}{8}$-inch rise.
No. 4. Rectangular obstruction, 3-inch base, $\frac{9}{16}$-inch rise.
No. 5. Rounded obstruction, 3-inch base, $1\frac{1}{2}$-inch rise.
No. 6. Rounded obstruction, 3-inch base, $\frac{3}{4}$-inch rise.
No. 7. Inclined plane, 30-inch ramp, $1\frac{1}{2}$-inch rise.
No. 8. Rectangular obstruction, 3-inch base, $\frac{7}{8}$-inch risc.

The impact forces developed under the various combinations of test conditions (i. e., truck, tire, load speed, and obstruction conditions) were computed in terms of the particular static wheel load involved. From the mass of data available, 43 conditions of load speed, and obstruction were selected as representative as set forth in detail in Table 1. The averages o these selected conditions were used in comparing the impact behavior of the various tire and truck combinations.

Table 1.—*Details of the 43 test conditions which are included in the averages used for comparison*

Speed (m. p. h.)	Load	Obstruction	
		Shock at—	Drop after—
7½ 12½ 17½	100 per cent tire capacity.	¾ by 3 inch rounded. ⅞ by 3 inch rectangular.	¾ by 3 inch rounded. 1½ by 30 inch inclined plane. ⅞ by 3 inch rectangular.
12	Empty truck. 75 per cent tire capacity. 100 per cent tire capacity. 150 per cent tire capacity.	¾ by 3 inch rounded. ⅞ by 3 inch rectangular.	¾ by 3 inch rounded. 1½ by 30 inch inclined plane. ⅞ by 3 inch rectangular.
12	100 per cent tire capacity.	¾ by 3 inch rounded. 1½ by 3 inch rounded. $\frac{9}{16}$ by 3 inch rectangular. ⅞ by 3 inch rectangular. 1⅛ by 3 inch rectangular.	¾ by 3 inch rounded. 1½ by 3 inch rounded. $\frac{13}{16}$ by 30 inch inclined plane. 1½ by 30 inch inclined plane. 1$\frac{15}{16}$ by 30 inch inclined plane. $\frac{9}{16}$ by 3 inch rectangular. ⅞ by 3 inch rectangular. 1⅛ by 3 inch rectangular.

PHYSICAL CHARACTERISTICS OF THE TIRES DESCRIBED

Only typical data for representative tires are included in this report as inclusion of all of the data concerning the physical characteristics of all of the tires tested would add unduly to the length without adding information which would be important in the discussion which follows. Figure 3 shows static load-deflection curves for these representative tires, and Table 2 gives the physical dimensions and weights of each at the various stages of the cutting-down process. Table 3 gives data obtained in the load-deflection tests. It will be noted that Table 3 includes measured areas of contact and computed unit load values. The load per unit of area as given in this table is an average value found by dividing the total load by the area of contact. This figure is not particularly significant since actually the load intensity is variable over the entire area of contact. However, the values given are useful as they do give some idea of the unit pressures which occur under the different tires for various static loads.

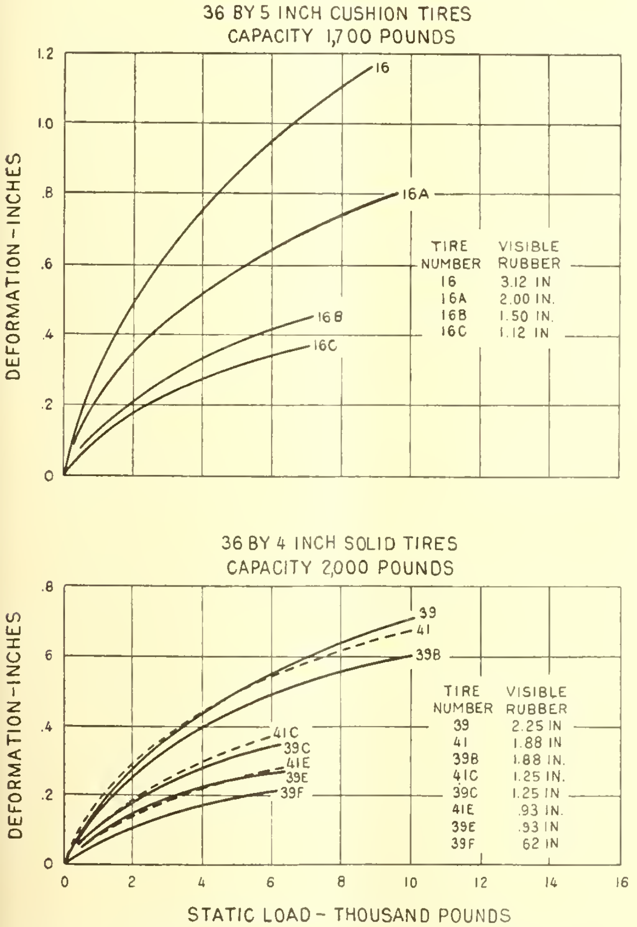

Figure 3.—Curves Showing Relation Between Load and Deformation for Various Tires

Table 2.—*Dimensions of tires*

Type and size	Tire test No.	Visible rubber	Face width	Weight
		Inches.	*Inches.*	*Pounds.*
Hollow center cushion, 36 by 5 inches	16	3.12	3¾	140
	16a	2.00	4	120
	16b	1.50	4$\frac{3}{16}$	111
	16c	1.12	4½	101
High profile solid, 36 by 4 inches	39	2.25	2¾	97
	39b	1.88	2¾	95
	39c	1.25	3$\frac{1}{16}$	82
	39e	.93	3⅛	79
	39f	.62	3$\frac{1}{16}$	72½
Regular solid, 36 by 4 inches	41	1.88	2⅞	94
	41c	1.25	3	88
	41e	.93	3⅛	80

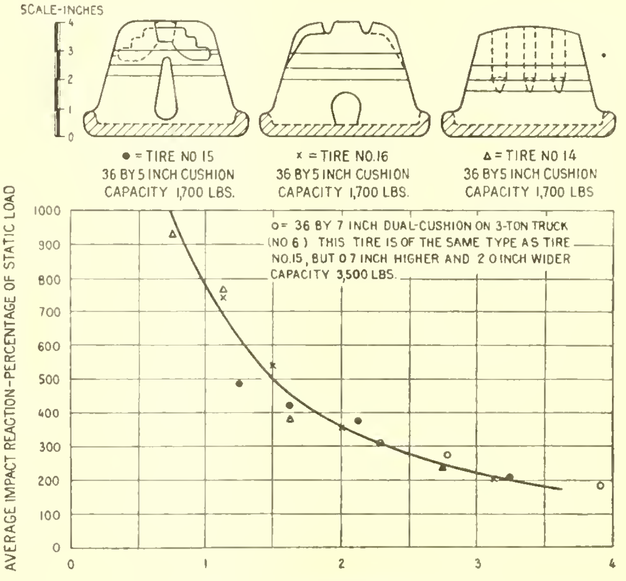

Figure 4.—Relation Between Tire Height and Average Impact Reaction for a 36 by 5 Inch Dual Cushion Tire on a 2-Ton Truck

TEST DATA DISCUSSED

Representative data from impact tests on some of the tires are given in Figures 4, 5, and 6. These curves show the effect of tread rubber thickness on the magnitude of the impact reaction, other factors being the same. The ordinates selected for the presentation of the data in this form are measured values, averaged from a variety of load, speed, and artificial obstruction conditions as previously explained.

Figure 4 shows these data for the three types of 36 by 5-inch dual cushion tires on a 2-ton truck, and also

TABLE 3.—*Results of static tests*

36 BY 5 INCH HOLLOW CENTER CUSHION

Tire test No.	Static load		Deflection	Contact			Average unit load	
	Percentage of capacity	Pounds		Width	Length	Area	Per inch of width	Per square inch of area
			Inches	*Inches*	*Inches*	*Sq. in.*	*Pounds*	*Pounds*
16	100	1,700	0.42	3.35	7.83	15.28	507	111
	200	3,400	.68	3.69	9.79	22.45	922	151
	500	8,500	1.13	4.53	12.93	37.50	1,876	227
16a	100	1,700	.31	4.10	6.67	26.56	415	64
	200	3,400	.48	4.13	8.63	30.76	824	110
	500	8,500	.75	4.90	10.66	44.60	1,735	191
16b	100	1,700	.19	4.30	5.97	20.52	395	83
	200	3,400	.30	4.52	7.40	25.07	753	136
	333	5,670	.40	4.80	8.30	34.81	1,181	163
16c	100	1,700	.16	4.66	5.28	20.85	366	82
	200	3,400	.25	4.78	6.12	27.15	716	125
	333	5,670	.33	4.95	7.05	32.10	1,145	177
36 BY 4 INCH HIGH-PROFILE SOLID								
39	100	1,700	0.24	2.48	5.91	12.62	685	135
	200	3,400	.38	2.61	7.30	16.40	1,303	207
	500	8,500	.65	3.16	9.60	25.73	2,690	330
39b	100	1,700	.22	2.91	5.65	14.45	586	118
	200	3,400	.35	3.01	7.02	18.93	1,133	180
	500	8,500	.58	3.38	9.37	28.75	2,528	296
39c	100	1,700	.16	3.12	5.05	12.93	545	132
	200	3,400	.26	3.23	6.35	17.58	1,052	194
	333	5,670	.34	3.42	7.19	21.40	1,660	265
39e	100	1,700	.13	3.40	4.90	13.12	500	129
	200	3,400	.20	3.50	5.70	16.95	972	201
	333	5,670	.26	3.65	6.50	20.53	1,553	276
39f	100	1,700	.10	3.50	3.82	11.66	486	146
	200	3,400	.16	3.58	4.53	14.39	950	236
	333	5,670	.21	3.72	5.16	17.40	1,525	325
36 BY 4 INCH REGULAR SOLID								
41	100	1,700	0.26	2.72	5.82	14.04	625	121
	200	3,400	.41	2.98	7.35	18.32	1,164	186
	500	8,500	.62	3.30	9.27	26.76	2,575	318
41c	100	1,700	.17	3.08	5.27	13.64	553	125
	200	3,400	.27	3.17	6.36	17.90	1,072	190
	333	5,670	.36	3.30	7.33	21.20	1,720	268
41e	100	1,700	.18	3.18	4.90	12.59	535	135
	200	3,400	.20	3.28	5.95	16.42	1,035	207
	333	5,670	.26	3.48	6.77	19.90	1,628	285

includes limited data for a 7-inch tire on a 3-ton truck, for reasons that will be discussed later. Figure 5 shows similar data for 36 by 4 inch dual solid tires of the regular and high-profile types, on the 2-ton truck. Figure 6 shows the same type of data for 36 by 5-inch dual high-profile tires on a 3-ton truck. Figure 8 shows another grouping of the data which indicates the increase in the impact reaction (expressed as a percentage of the static reaction) as the height of visible rubber is reduced by definite percentages of its original height. The tires from which these data were obtained are indicated in this figure.

CUSHION TIRES

The data presented in Figure 4 were derived principally from the tests in which three types of 36 by 5-inch dual cushion tires were used on the 2-ton truck. The curve was derived from the points obtained from the tests on the 36 by 5-inch tires and may be said to represent fairly the average relation found for these three types of tires. This curve indicates that if 1 inch of tread rubber is lost from a new tire with an original height of visible rubber of 3⅛ inches, the magnitude of the average impact reaction is increased from 215 to 340 per cent of the static load. A further reduction of 1 inch in tread rubber thickness increases the average impact reaction to some 700 per cent of the static load.

Stated in another way, the curve indicates for these tires that, if 32 per cent of the visible height of tread rubber of the new tire is removed, the impact reaction is increased 1.6 times and that if 64 per cent of the height of visible rubber is removed the average impact reaction is some 3.2 times that for a new tire under the same conditions. Or, viewed from another angle, this curve indicates that for this "average" 36 by 5 inch cushion tire, the removal of about 1⅜ inches (about 44 per cent) of the original tread rubber thickness will cause the impact reactions to be doubled while if 1⅞ inches (about 60 per cent) of tread rubber is removed the impact reaction is trebled, and quadrupled if 2¼ inches is removed (about 72 per cent).

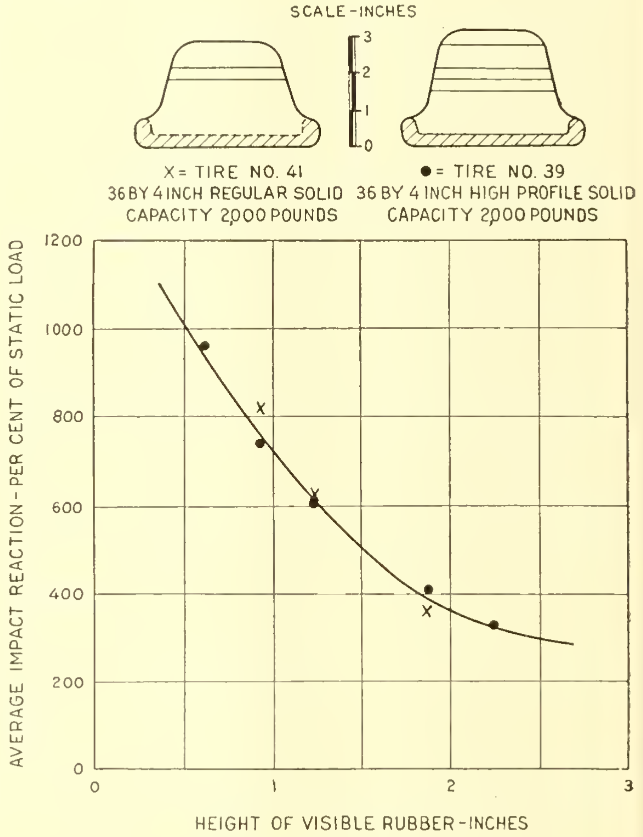

FIGURE 5.—RELATION BETWEEN TIRE HEIGHT AND AVERAGE IMPACT FOR A 36 BY 4 INCH SOLID TIRE ON A 2-TON TRUCK

Figure 4 shows three points representing data obtained from tests on dual 36 by 7-inch cushion tires. Although the number of tests made on cut-down cushion tires larger than 5 inches was very limited, the data obtained are valuable since when compared with those obtained with the 5-inch tires, as in Figure 4, there is no indication that the relation between the height of visible rubber and the magnitude of the impact reaction (expressed as a percentage of static load) is different from that established for the 5-inch tires. While the 7-inch tires were not run at the same load in pounds as the 5-inch tires, the loads expressed as a percentage of tire capacity, the speeds, and the obstructions were all practically the same.

SOLID TIRES

Figure 5 shows similar data for the dual 36 by 4-inch solid tires of the regular and high-profile types when used on the 2-ton truck. From these data it appears that if 2⅛ inches be taken as the height of visible rubber of a 36 by 4-inch new solid tire, the average impact is doubled if 1 inch of tread rubber (47 per cent) is removed, and trebled if 1⅝ inches (76 per cent) is removed.

From the data on 36 by 5-inch high-profile tires on a 3-ton truck (fig. 6), it is indicated that if 2⅜ inches be taken as the normal height of visible rubber, the impact reaction of a new tire is doubled if about 1⅛ inches (47 per cent) of tread rubber is removed and trebled if about 1⅝ inches (68 per cent) is removed.

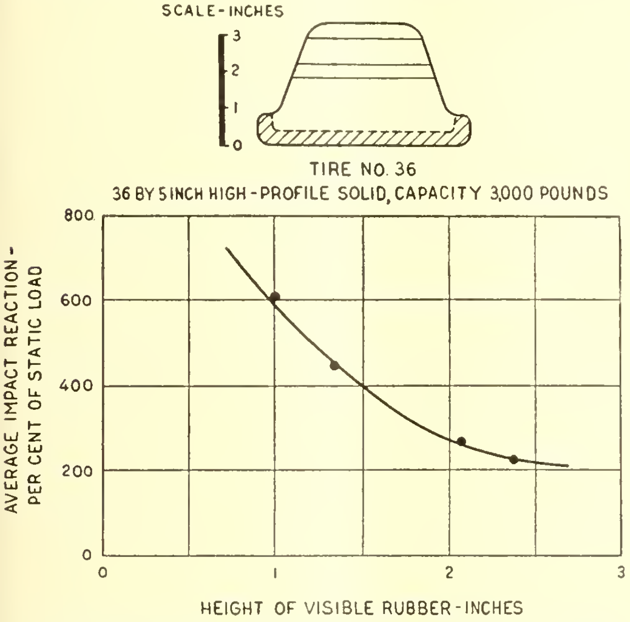

Figure 6.—Relation Between Tire Height and Average Impact Reaction for a 36 by 5 Inch Dual Solid Tire on a 3-Ton Truck

The data presented in Figures 4, 5, and 6 bring out clearly the very important influence of tread-rubber thickness on the cushioning properties of tires of both the cushion and the solid types.

CUSHION TIRES VERSUS SOLID TIRES

Before attempting a comparison of the relative cushioning properties of tires of the cushion and solid types, there are certain points concerning their use which should be thoroughly understood. Reference is made to the full discussion of the effect of tire width contained in the earlier report in which it was brought out that, in general, a wide tire tends to cause heavier impact reactions than a narrow tire, other conditions being the same. This conclusion is of immediate importance since in practice when a solid tire is replaced by a cushion tire it is necessary to increase the width because of the differences in the rated carrying capacities of the two types. For example, a 5-inch cushion tire is, in practice, considered to be the replacement size for a 4-inch solid tire. The rated capacity of a 5-inch cushion tire is 1,700 pounds, of a 4-inch solid tire 2,000 pounds and of a 5-inch solid tire 3,000 pounds. Briefly, when we replace a solid tire with one which by its construction has better cushioning properties we are forced to use a wider tire and in so doing must sacrifice at least some of the benefits gained.

A comparison of Figures 4 and 5 might lead to the conclusion that there is no real difference between the behavior of a cushion tire and that of a solid tire of the same height. Let us consider this point. In the first place, it should be noted that the impact reactions are given as a percentage of static load. As the capacity loads of the tires were not the same and as the tires of both types were loaded in each instance to their respective capacity loads or to some definite percentage of capacity load, the curves do not give comparable impact reactions in pounds. They simply show the relative increase in impact reaction for corresponding losses in the thickness of the tread rubber on the particular tire types under consideration.

By virtue of the construction of a cushion tire, it may naturally be expected to yield lower impact values than a solid tire of the same width and with the same amount of visible rubber. As has been brought out, a cushion tire is not comparable with a solid tire of the same width because of the difference in carrying capacity.

Referring to Figure 7 (reprinted from the earlier report), it is shown that on a 2-ton truck an increase in tire width of 50 per cent (from dual 4-inch to dual 6-inch) caused an increase in the impact reaction of about 30 per cent (at 12 miles per hour under the given test conditions). It appears reasonable to suppose, therefore, that as the 5-inch cushion tires upon which Figure 4 is based are 25 per cent wider than the 4-inch tires upon which Figure 5 is based, the impact values of the former (the 5-inch cushion) are about 10 to 20 per cent higher than they would have been if 4-inch cushion tires could have been used.

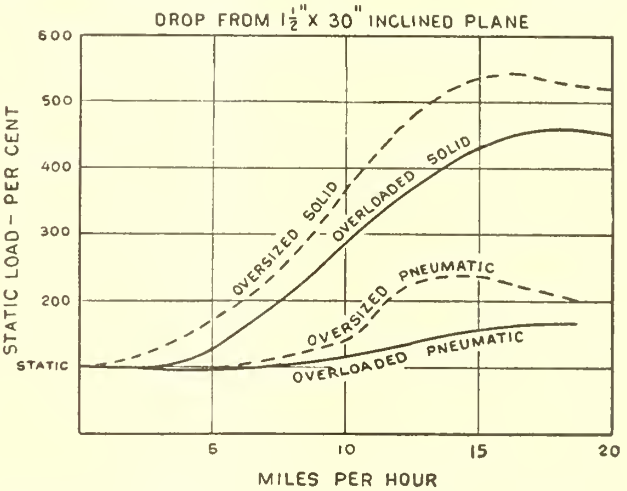

Figure 7.—Effect of Tire Size on Impact Reactions Using a 2-Ton Truck. The Overloaded Tires Carried a Load Equal to 150 Per Cent of Their Capacity and the Oversized Tires Carried the Same Load as the Corresponding Overloaded Tires. Curve Marked "Oversized Solid" Represents Dual 36 by 6 Inch New Regular Solid Tires, and the One Marked "Overloaded Solid" Represents Dual 36 by 4 Inch New Regular Solid Tires

If this reasoning is sound, it is evident that if solid tires could be replaced by cushion tires on an equal width basis, the latter would be some 10 to 20 per cent better than the former at any given height. As shown in the figures (on an equal capacity basis), there is very little difference between the two types at any given height. These data, then, indicate that the advantage of using cushion tires may be partially offset and, in the examples illustrated in Figures 4 and 5, completely offset by the disadvantage of having to use wider cushion tires in order to carry loads comparable to those which may be carried by replacement tires of the solid type. It is important to remember that this statement applies only to tires having equal heights of visible rubber.

In view of the foregoing one might reasonably ask: "Why use cushion tires?" The principal reasons may be stated as follows:

1. The average height of visible rubber for new cushion tires is about 1 inch or nearly 50 per cent greater than that of the same nominal size of regular solid tire and somewhat greater than this for the smaller size of solid tire for which the cushion is a replacement. As pointed out in the earlier report the thicker the tread (i. e., the greater the thickness of visible rubber) the greater the cushioning quality.
2. Although the use of cushion tires necessitates wider tires, the disadvantage in the use of wider tires (as brought out in the earlier report) is offset by whatever cushioning qualities may be inherent in cushion tires. Moreover, for a given force (static or impact) the load concentration on the pavement is less for wide tires than for narrow tires, and while this may not be of primary importance on the higher types of road, it undoubtedly is of considerable importance on the softer types.
3. The possible loss in height by cushion tires through wear in practical use is generally limited by the size, shape, and position of the structural cavities. It will be pointed out in the subsequent discussion that this tends to act as a safeguard to both the vehicle and the pavement.

Cushion tires of the circumferential or annular hollow-center types, such as were included in these tests, are not generally considered suitable for use at high speeds after wear of the tread has exposed the internal cavity and thus altered the structural action of the tire. The use of tires of this type is thus practically limited to the amount of wear which can take place before this condition obtains and is dependent upon the position, size, and shape of the internal cavity. In this way a type whose internal cavity becomes exposed while there is still a considerable amount of visible rubber on the tire acts as a safeguard to the highway because in such a condition of wear it is unsafe to use at speeds such as would make it objectionable from the standpoint of impact.

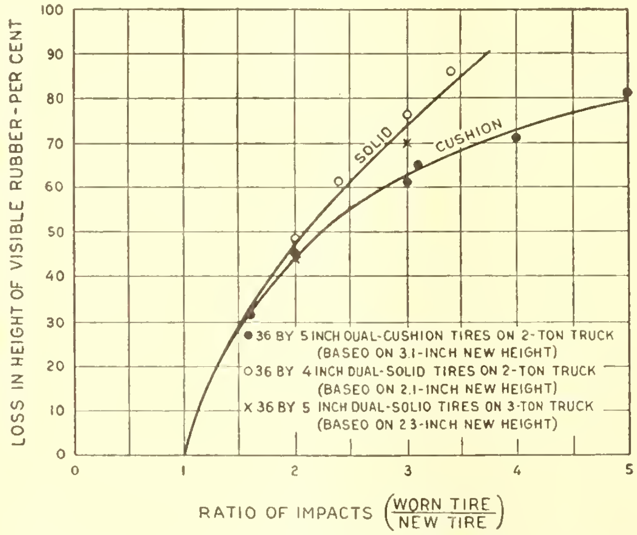

Figure 8.—Comparison of Ratio of Impact of Worn Tire to New Tire with Loss in Height of Visible Rubber

Table 4.—*Tire conditions and results of impact tests*

36 BY 5 INCH DUAL CUSHION TIRES ON 2-TON TRUCK

[For new tire, over-all height 4 inches, visible rubber 3.1 inches, average impact 215 per cent of static load]

Over-all height of worn tire	Loss in tread thickness	Loss in over-all height	Average impact of tire, percentage of static load	Ratio of impact of worn tire to new tire	Visible rubber	Loss in visible rubber
Inches	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Inches*	*Per cent*
3.0	1.0	25	345	160	2.1	32
2.6	1.4	35	430	200	1.7	45
2.1	1.9	48	645	300	1.2	61
2.0	2.0	50	715	330	1.1	65
1.8	2.2	55	860	400	0.9	71
1.6	2.4	60	1,025	480	0.7	77

36 BY 4 INCH DUAL SOLID TIRES ON 2-TON TRUCK

[For new tire, over-all height 3 inches, visible rubber 2.1 inches, average impact 340 per cent of static load]

2.0	1.0	33	680	200	1.1	48
1.7	1.3	43	820	240	0.8	62
1.4	1.6	53	1,020	300	0.5	76
1.2	1.8	60	1,140	340	0.3	86

36 BY 5 INCH DUAL SOLID TIRES ON 3-TON TRUCK

[For new tire, over-all height 3.2 inches, visible rubber 2.3 inches, average impact 240 per cent of static load]

2.2	1.0	31	480	200	1.3	44
1.6	1.4	44	720	300	0.7	70

There is no apparent reason why cushion tires worn to the limit of visible rubber permitted for such tires should not assume the status of solid tires and then be worn to the limit of visible rubber permitted for this type. If tires should be or are discarded by a truck owner as a matter of maintenance of equipment when the impact reaction has increased to a point where it is a certain number of times the impact reaction of the new tires, and this factor be taken as three, then 5-inch cushion tires would be removed after the visible rubber had been reduced to about 1¼ inches (about 40 per cent of the original visible height) and 4-inch and 5-inch solid tires would be removed after the visible rubber had been reduced to about ½ inch and ¾ inch, respectively (24 and 32 per cent of the original visible heights).

APPLICATION OF DATA TO ACTUAL PROBLEMS

The data taken from the curves to support the above conclusions have been listed in Table 4, and are shown graphically in Figure 8. Before applying the interpretations to actual problems, it is again pointed out that the impact in pounds is not the quantity used in comparing the reactions of tires. The quantity used is the increase in reaction of a given new tire when loss occurs in the height of visible rubber. Comparison of the curves representing the action of solid tires and cushion tires indicates that for a given percentage of loss in visible rubber the cushion tires show a greater percentage increase in impact reaction. This is not unreasonable when it is remembered that, for a given truck, the proper cushion tire would be about 1 inch thicker than the replacement solid tire. A given percentage loss in visible rubber would therefore mean a greater loss in inches on cushion tires than on the corresponding solid tires. A new cushion tire should cause lower impact reactions than the same size of new solid tire because of differences in height and structure, and the impact reactions should become more nearly equal as the tires approach a worn-out condition because of a gradual approach to the same physical condition of

(Continued on p. 152)

THE INTERRELATED EFFECTS OF LOAD, SPEED, TIRES, AND ROAD ROUGHNESS ON MOTOR TRUCK IMPACT[1]

Reported by **JAMES A. BUCHANAN**, Associate Engineer of Tests, Division of Tests, Bureau of Public Roads[2]

Since the tests reported in this article were conducted, a thorough investigation of the accuracy of the instruments used has been made by the United States Bureau of Standards, in cooperation with the United States Bureau of Public Roads. A summary account of this work was published in Public Roads, July, 1930. The committee which guided the investigation recommended that the following statement regarding the accuracy of the data, accompany this report:

The results obtained in the motor-truck impact tests are, in general, systematically from 10 to 15 per cent too low. A conservative estimate of the dispersion of the measured values of acceleration indicates that 90 per cent of the significant accelerations lie within less than 15 per cent of the mean, for tests involving artificial obstructions. For tests on natural road surfaces the exact value of the dispersion is uncertain.

THE data obtained in the earlier motor truck impact tests shows that the magnitude of impact reaction is dependent on at least four major variables which are: Road roughness, tire equipment, wheel load, and vehicle speed. A graphic representation of the interrelationship of these four factors, based on the data obtained on the artificially roughened test road at Arlington, Va., is shown in the form of an isogram chart in Figure 1. In this figure curves of equal impact reaction show the impact force which may reasonably be expected to occur with any combination of tire equipment, wheel load and truck speed on the particular road under consideration and within the limits used in obtaining the data. The figure contains four groups of curves, each group representing one of the four representative tire types used in the tests. Figure 1 is of no practical use because it is based on a peculiar condition of surface roughness, artificially obtained, but it is significant because it indicates the possibility of conducting tests on actual road surfaces and collecting data which, when shown in a similar way, would be of considerable practical importance.

This possibility was laid before the cooperative committee in October, 1926, together with a tentative program of tests which would involve desired ranges of road surface roughness, tire equipment, wheel load, and vehicle speed, and the proposed program was authorized. The details of the test program which was followed are described in the following text.

TEST TRUCKS AND INSTRUMENTS DESCRIBED

The 2-ton and 5-ton test trucks were equipped with the coil spring accelerometer developed by the Bureau of Public Roads. This equipment had been used in previous impact tests and the method of obtaining impact forces from the data has been described in an earlier publication.[3]

The general specifications of the test trucks are given in Table 1 and the general characteristics of the truck tires and of the accelerometer elements which were used are given in Table 2.

In order to insure consistency of action, the rear springs of the trucks had been thoroughly cleaned, greased and provided with jackets during earlier work and they remained in satisfactory condition throughout the tests.

It was necessary to have the entire equipment mobile and a field office with platforms for loading and storage was built on a trailer chassis. A service truck carrying a chain hoist on a platform body was used for towing the trailer and to carry wheels, tires, gasoline, and other supplies necessary for the operation of the test trucks.

Each set of tires was mounted permanently on wheels in order that the necessary tire changes might be quickly and conveniently made merely by substituting wheels. This arrangement had the further advantage of insuring that tire installations on the trucks could be faithfully repeated regardless of the number of times the wheels were interchanged. The types selected were the same as those upon which the preliminary isogram had been based, namely, pneumatic, new cushion, new regular solid, and worn solid. These tire selections represent the range in equipment ordinarily found on the highways to-day and are briefly described as follows: Pneumatic—high pressure pneumatic cord tires on demountable rims; new cushion—new, hollow-center, pressed-on cushion tires of average height with nonskid, cushioning tread design; new regular solid—new, pressed-on, regular solid tires of average height and with smooth tread (the present tendency is to call such tires low-profile solids), worn solid--solid tires which had been cut down from new stock on a special tire-cutting machine and were thus true and round. Tires which have been worn down in service to heights corresponding to those of the worn tires as used in these tests have generally been subjected to aging and hardening influences, and service-worn tires are seldom, if ever, true and round. Figure 2 shows the tires mounted dually on wheels and ready for use. Figure 3 shows the actual cross sections of the individual tires.

Some of the details of the behavior of single tires under static load are given in Table 3. The term "visible rubber height" as herein used refers to the radial thickness of tread rubber from the outside of the steel flange of the tire rim to the extreme outer (tread) surface of the rubber, the measurement being

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[1] These tests are a part of the investigation of motor truck impact being conducted by the Bureau of Public Roads and were made with the cooperation of the Rubber Manufacturers Association and the Society of Automotive Engineers.

[2] Acknowledgment is made of the assistance of J. W. Reid, formerly of the Rubber Association, in planning the tests and obtaining the data.

[3] See Public Roads, vol. 7, No. 4, June, 1926, and vol. 11 No. 5, July, 1930.

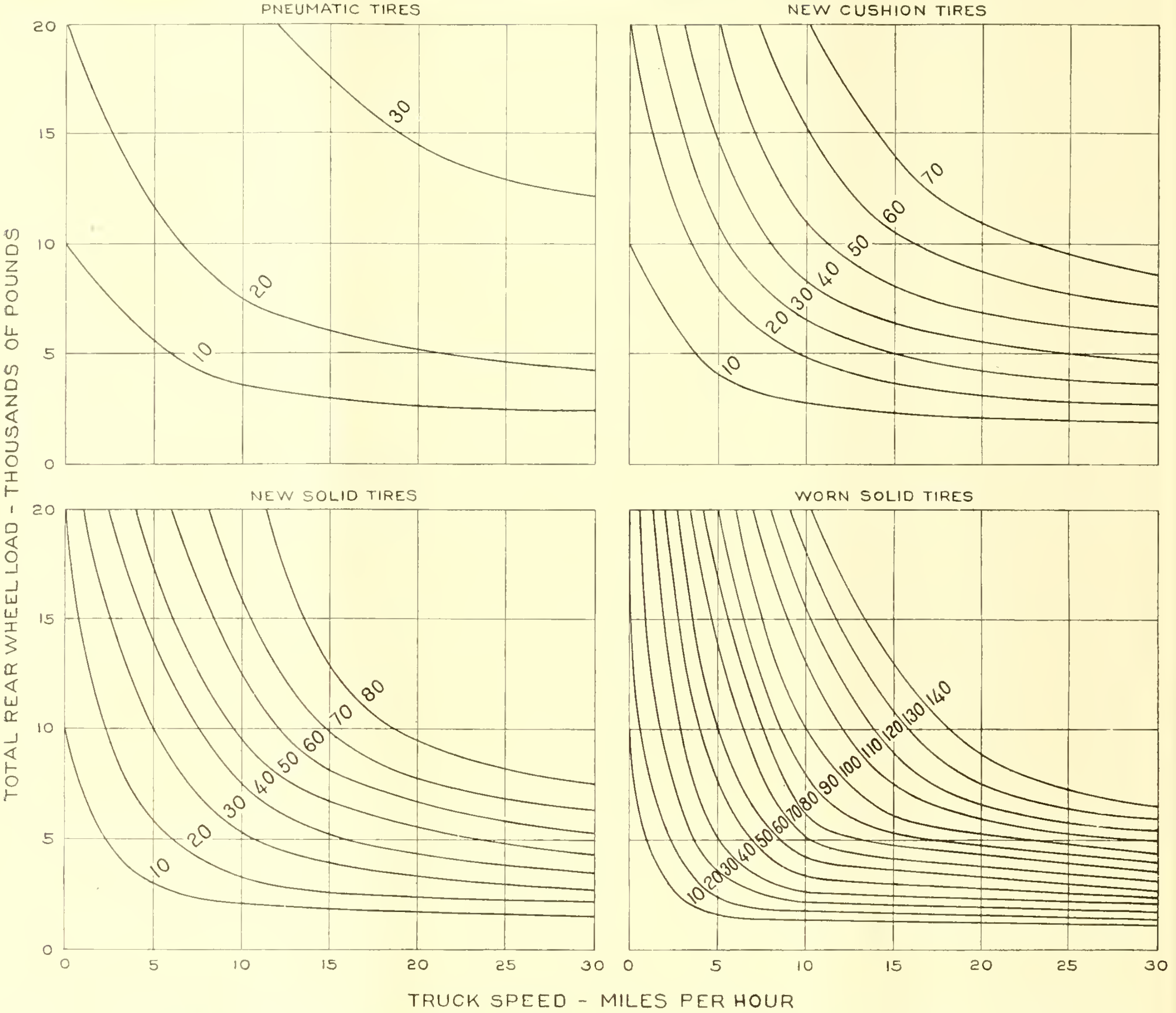

Figure 1.—Isodynamic Curves for Arlington Farm Test Road With Artificial Obstructions for Determination of Maximum Total Vertical Reactions in Thousands of Pounds

taken at zero load. The average static load-deflection curve for each tire type is shown in Figure 4, the deflections under a static load of 10,000 pounds being approximately 2.5 inches, 1 inch, 0.7 inch, and 0.3 inch, respectively, for the pneumatic, new cushion, new solid, and worn solid tire types.

At the beginning of the tests, the rear wheel loads were standardized at 2,500, 5,000, 7,500, and 10,000 pounds. The 2,500 and 5,000 pound loads were run with the 2-ton truck, and the 7,500 and 10,000 pound loads were run with the 5-ton truck. The light load of each truck was accurately measured on stationary platform scales and securely fastened in position. Then the heavier load (also weighed on the platform scales) was built up on each truck by adding 100-pound lead or iron weights. The positions of these extra weights were marked on the trucks so that wheel loads might be duplicated readily and with exactness. Tables 4 and 5 show the general characteristics of the loadings.

TESTS COVERED WIDE RANGE IN TRUCK SPEEDS AND COND TION OF ROAD SURFACE

The trucks were operated over the test sections at speeds varying by small increments from the minimum up to the maximum obtainable. The average speed of each run was computed from stop-watch observations and the known lengths of the test sections (one-twentieth mile). Variations in speed were observed on speedometers mounted on the trucks. Runs varying from the average speed by more than one-half mile per hour, as registered by the speedometers, were discarded.

The highway sections used in these tests were selected as representative in type or roughness. They were marked in one-twentieth mile lengths and points were spotted on the pavements to guide the trucks over the test sections. Letter symbols were used to designate the various highways, as shown in Table 6, and the test sections were all straight, nearly level, and with but slight crowns.

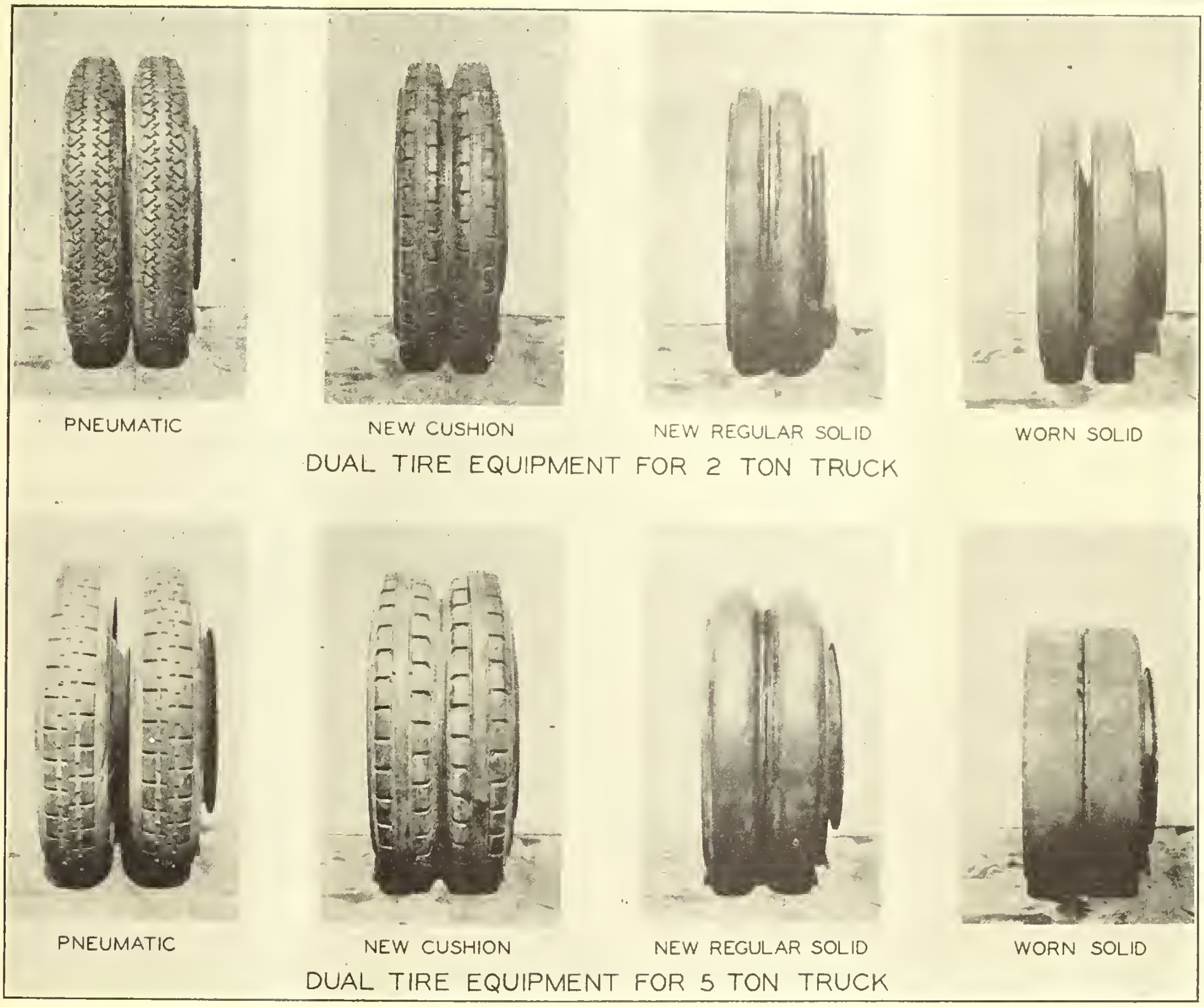

FIGURE 2.—REPRESENTATIVE TIRE EQUIPMENTS SELECTED FOR TESTS

TABLE 1.—*Truck specifications (empty)*

Capacity of truck ..... tons	2	5
Truck symbol	B	A
Wheel base ..... feet	13.55	13.00
Normal tread width:		
Rear ..... do	5.45	5.42
Front ..... do	4.95	5.30
Overall:		
Length ..... do	20.3	19.2
Width ..... do	7.2	7.4
Inside body:		
Width ..... do	6.5	6.3
Length ..... do	12.0	11.5
Inside end of body to rear axle ..... do	4.65	2.15
Height of body floor:		
Rear ..... do	[1] 4.08	[2] 4.03
Front ..... do	[1] 3.87	[2] 3.78
Tire equipment, front	([3])	([4])
Weight of truck:		
Total ..... pounds	[1] 7,670	[2] 11,423
Front axle ..... do	2,940	5,075
Rear axle ..... do	[1] 4,730	[2] 6,348
Sprung weight, rear axle ..... do	3,250	3,900
Unsprung weight, rear axle ..... do	[1] 1,480	[2] 2,448
Number of leaves in rear spring	10	17
Number of leaves in front spring	9	15
Total spring thickness:		
Rear ..... feet	.32	.60
Front ..... do	.23	.41
Spring width:		
Rear ..... do	.29	.29
Front ..... do	.21	.20
Spring length:		
Rear ..... do	4.47	4.47
Front ..... do	3.30	3.55
Spring camber:		
Rear ..... do	.14	.16
Front ..... do	.23	.16
Approximate spring rate per inch, rear ..... pounds	1,200	2,000

[1] Based on dual 36 by 4 inch new solid tires on rear wheels (tire No. 40).
[2] Based on dual 36 by 6 inch new solid tires on rear wheels (tire No. 37).
[3] 35 by 5 single pneumatic 65-pound pressure.
[4] 34 by 6 single nonskid solid.

TABLE 2.—*Test data*

Tire type, truck, and test symbol	Pneumatic		New cushion	
	2-ton 47-B	5-ton 49-A	2-ton 16-B	5-ton 7-A
Accelerometer:				
Spring weight combination	2a.6.H	3a.B.L	2a.6.0	3a.B.C
Free period of vibration, seconds	0.06192	0.05640	0.03351	0.03693
Calibration factor, feet per second per second per inch	429	517	1,496	1,275
Unsprung weight per rear wheel, pounds	740	1,190	820	1,380
Unsprung mass per rear wheel	23.0	37.0	25.5	42.8
Mass multiplied by calibration factor	9,870	19,130	38,150	54,570
Tire size, dual mounting, inches	36 by 6	38 by 7	36 by 5	36 by 7
Tire capacity:				
Light load, per cent	57	125	73	107
Heavy load, per cent	114	167	147	143

Tire type, truck, and test symbol	New solid		Worn solid	
	2-ton 40-B	5-ton 37-A	2-ton 39f-B	5-ton 31f-A
Accelerometer:				
Spring weight combination	2a.2.L	3a.D.L	2a.2.F	3a.D.F.
Free period of vibration, seconds	0.02539	0.02542	0.01983	0.02004
Calibration factor, feet per second per second per inch	2,655	2,654	4,604	4,523
Unsprung weight per rear wheel, pounds	740	1,224	751	1,115
Unsprung mass per rear wheel	23.0	38.0	23.3	34.6
Mass multiplied by calibration factor	61,060	100,850	107,270	156,500
Tire size, dual mounting, inches	36 by 4	36 by 6	36 by 4	36 by 6
Tire capacity:				
Light load, per cent	62	89	62	89
Heavy load, per cent	125	119	125	119

TABLE 3.—*Tire data*

Test No.	Load: Percentage of capacity	Load: Actual	Deflection	Contact: Width	Contact: Length	Contact: Area	Average unit load: Per inch of width	Average unit load: Per square inch of area	Air pressure	Nominal size	Visible rubber height	Width: Top	Width: Bottom
PNEUMATIC TYPE													
		Lbs.	*In*	*In.*	*In.*	*Sq. in.*	*Lbs.*	*Lbs.*	*Lbs./sq. in.*	*In.*	*In.*	*In.*	*In.*
47	100	2. 200	.72	4. 01	8. 73	13. 65	549	118	90	36 by 6	[1] 7⅛	4 1/16	6 21/32
	200	4, 400	1. 27	4. 42	11. 47	39. 24	996	112	92				
	500	11, 000	2. 80	6. 00	17. 45	66. 04	1, 833	166	98				
49	100	3, 000	. 85	4. 90	9. 57	24. 54	612	122	100	38 by 7	[1] 8	5	7 17/32
	200	6, 000	1. 51	5. 25	12. 36	46. 70	1, 143	129	101				
	500	15, 000	3. 34	7. 40	18. 85	80. 50	2, 027	186	108				
NEW CUSHION TYPE													
16	100	1, 700	. 42	3. 35	7. 83	15. 28	507	111		36 by 5	3⅛	3¾	5
	200	3, 400	. 68	3. 69	9. 79	22. 45	922	151					
	500	8, 500	1. 13	4. 53	12. 93	37. 50	1, 876	227					
7	100	3, 500	. 48	5. 30	9. 75	29. 50	660	119		36 by 7	3¼	5⅜	7 1/16
	200	7, 000	. 74	5. 77	11. 60	42. 42	1, 213	165					
	500	17, 500	1. 21	6. 78	14. 38	62. 88	2, 581	278					
NEW SOLID TYPE													
40	100	1, 700	. 28	2. 43	7. 04	15. 43	700	110		36 by 4	2	2¾	3 11/16
	200	3, 400	. 43	2. 83	8. 31	20. 84	1, 201	163					
	500	8, 500	. 69	3. 48	10. 40	30. 25	2, 442	281					
37	100	3, 500	. 45	3. 83	7. 80	26. 75	914	131		36 by 6	2¼	4 5/16	5⅝
	200	7, 000	. 61	4. 40	9. 32	35. 65	1, 591	196					
	500	17, 500	. 89	5. 38	11. 53	52. 75	3, 252	332					
WORN SOLID TYPE													
39f	100	1, 700	. 10	3. 50	3. 82	11. 66	486	146		36 by 4	¾	3 7/16	3¾
	200	3, 400	. 16	3. 58	4. 53	14. 39	950	236					
	333	5, 670	. 21	3. 72	5. 16	17. 40	1, 526	325					
31f	100	3, 500	. 16	5. 47	4. 97	21. 95	640	176		36 by 6	⅝	5 5/16	5⅛
	200	7, 000	. 22	5. 66	5. 90	27. 25	1, 236	258					
	333	11, 670	. 27	5. 88	6. 34	32. 00	1, 980	365					

[1] Over-all sectional beigbt, fully inflated and including demountable rim.

The test road surfaces were calibrated with the relative roughness indicator [4] being developed by the bureau. This was carefully done at varying speeds, and the maximum and minimum roughness factors obtained at a car speed of 30 miles per hour were approximately 800 and 100 units per mile. The instrument was installed between the front axle and chassis frame of a large sedan automobile fully equipped with rebound snubbers and 6.50 by 20 balloon tires inflated to a pressure of 35 pounds.

Since the vehicle on which the device is mounted is an integral part of the apparatus, the data obtained with one vehicle are not directly comparable with those obtained with another. However, as the same vehicle was used to obtain data on each of a series of roads it is believed that the respective factors obtained indicate the relative roughness with reasonable accuracy.

The road sections had been selected and the impact tests substantially completed before the relative roughness indicator was available. It was later found that the surface roughness range could be adequately covered by five of the eight sections tested. Final computations were therefore made for the five roads only. The roughness calibration curves for the selected sections are shown in Figure 5.

Municipal authorities in the District of Columbia and in Alexandria, Va., were interested cooperators in the tests made within their jurisdictions. Traffic officers were assigned to assist in the work and they were extremely valuable in making it possible to conduct the tests on the public streets with safety. "No parking" signs were placed temporarily at several points to facilitate the work. Figure 6 shows the equipment used in these tests.

TABLE 4.—*Load data*

	2-ton truck		5-ton truck	
Rear wbeel total load……tons..	1¼	2½	3¾	5
Rear wheel, average unsprung load……pounds..	745	745	1, 200	1, 200
Rear wheel, average sprung load……do....	1, 755	4, 255	6, 300	8, 800
Rear axle load……do....	5, 000	10, 000	15, 000	20, 000
Front axle load……do....	3, 005	4, 255	7, 240	8, 490
Gross truck load……do....	8, 005	14, 255	22, 240	28, 490
Truck capacity……per cent..	8	164	109	171
Average ratio, $\frac{\text{Unsprung rear}}{\text{Sprung rear}}$	0. 425	0. 175	0. 191	0. 136
Average ratio, $\frac{\text{Unsprung rear}}{\text{Total, rear}}$	. 298	. 149	. 160	. 120

TABLE 5.—*Load data*

	Rear wheel load		Front wheel load
	Sprung	Total	Total
	Pounds	*Pounds*	*Pounds*
Average empty 2-ton truck	1, 625	2, 370	1, 470
Averago empty 5-ton truck	1, 950	3, 150	2, 532

TABLE 6.—*Data on test road surfaces*

Road	Condition	Type	Direction	Location
				Washington, D. C.:
A	Fair	Sheet asphalt	W. to E.	N Street, Second to First Street SW.
B	Good	Asphalt block	N. to S.	South Capitol Street, P Street to Potomac Avenue.
C	Fair	do	S. to N.	South Capitol Street, Potomac Avenue to P Street.
D	Good	Sheet asphalt	do	First Street, O Street to N Street SW.
E	Poor	Stone [1]	do	First Street, N Street to M Street SW.
F	Fair	Macadam	E. to W.	M Street, First Street to Second Street SW.
G	Poor	do	N. to S.	Delaware Avenue, M Street to N Street SW.
H	Good	Vitrified brick	W. to E.	Alexandria, Va., King Street, railroad bridge to Little River Road.

[1] Old stone road, neither cobble nor stone block in type. It was selected because it could be relied upon to maintain its roughness during these tests.

DATA CONSIDERED TO INDICATE ONLY GENERAL TRENDS BECAUSE OF LIMITATIONS OF THE TESTS

The data herein presented are based upon operations with the equipment described and it is desirable that the limitations which should be applied in their practical use be thoroughly understood.

Although the two trucks which were used are familiar commercial products, they differed from each other considerably in design and the loads on them varied widely when expressed in terms of truck capacity.

The data are influenced also by the particular tire equipments used. While these tires were selected as representative of their respective types at the time they were furnished, it is undoubtedly true that, due to the development in truck tire manufacture, better tires are being made to-day. Thus, in a sense, the tires used may be considered obsolete. Nevertheless, so far as type and condition are concerned, these tires do not depart widely from the general run of tires found in truck service at the present time. The data are applicable to current practice provided due allowances are made for changes in such major characteristics as tire dimensions.

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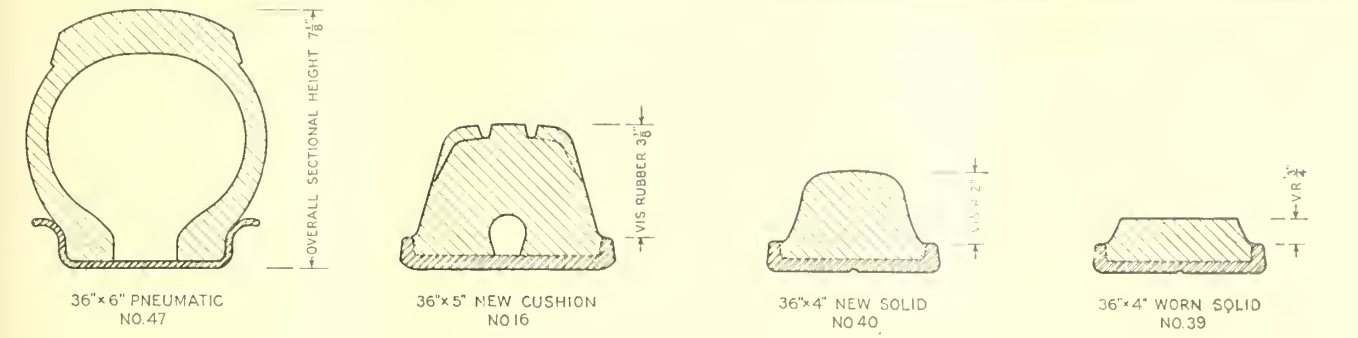
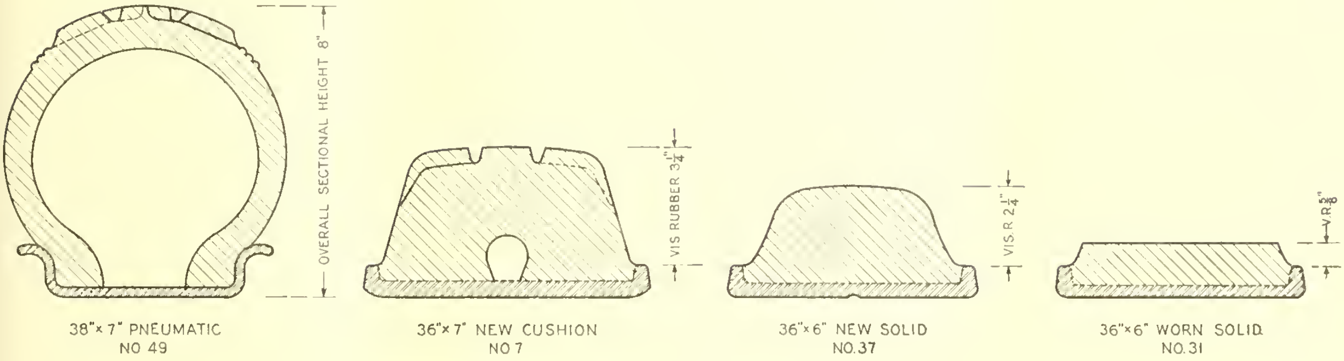
[4] See An Instrument for the Measurement of Relative Road Roughness, Public Roads, vol. 7, No. 7, September, 1926.



EQUIPMENT USED ON 2-TON TRUCK



EQUIPMENT USED ON 5-TON TRUCK

Figure 3.—Sections of the Representative Tires Selected for Tests

The principal limitation appears to be in the assignment of roughness qualities to the various road surfaces. Verbal descriptions are not adequate since, obviously, the same interpretation would not universally be made from them. Photographs were taken but were found unsatisfactory in bringing out differences. Detailed profiles might have been taken but the application of the data to actual problems would then have necessitated similar arduous processes in obtaining profiles for each and every application. The relative roughness indicator offered the most ready means for evaluating the surface roughness of the roads. It was recognized that the factors of roughness pertain only to the particular indicator installation with which they were obtained. They are relatively correct and sufficiently accurate with respect to one another to express the general differences between the various test sections.

In view of these limitations the data presented in this report are considered to indicate general trends of the interrelated effects of the four major variables under discussion.

METHOD OF EXPRESSING IMPACT VALUES DESCRIBED

It was decided that for each combination of the four variables the average of the five highest impacts occurring within the one-twentieth-mile test sections would be a reasonable and representative value to seek. For different combinations, then, of the four major variables—road, tire, load, and speed—this value represents the average magnitude of the greatest impact forces which might reasonably be expected to occur as frequently as one hundred times per mile. Although this was an arbitrary selection, it was felt that if the heavier impact reactions occurred on a given road at least as frequently as one hundred times per mile (an average of once every 50 feet) their average would be a representative criterion of that particular surface condition. Since the unsprung component of motor-truck impact is generally the major or critical quantity, it was considered sufficient to compute the unsprung impact component (determined by multiplying the unsprung mass by its acceleration) and to add to it the sprung component (determined by static weighing) in order to arrive at the total impact force developed by the rear wheel. Up to the present time, there has been no attempt to express the absolute maximum impact within the one-twentieth-mile section in terms of the average of the five highest impacts, but the data are available whenever the derivation of such an expression may be found desirable.

The impact reaction was plotted against truck speed for each of the test conditions and smooth curves were drawn through these plotted points. These curves were then read at regular intervals to obtain data for the construction of the isograms. The inclusion in this report of copies of all of the original records, the computation tabulations, and the preliminary work sheets therefrom would only make it unnecessarily extended.

EXAMPLE OF METHOD OF COMPUTATION PRESENTED

As an example of the method of constructing the isograms, the following detailed procedure is given for one of the intermediate conditions—new solid-tire equipment on the fair sheet asphalt surface.

For this, as in all of the selected combinations of tire equipment and road surface, runs for each of the four loads were made at speeds from about 3 miles per hour up to the maximum obtainable, in increments of about 3 miles per hour. For each run a record was obtained with the coil spring accelerometer used in previous

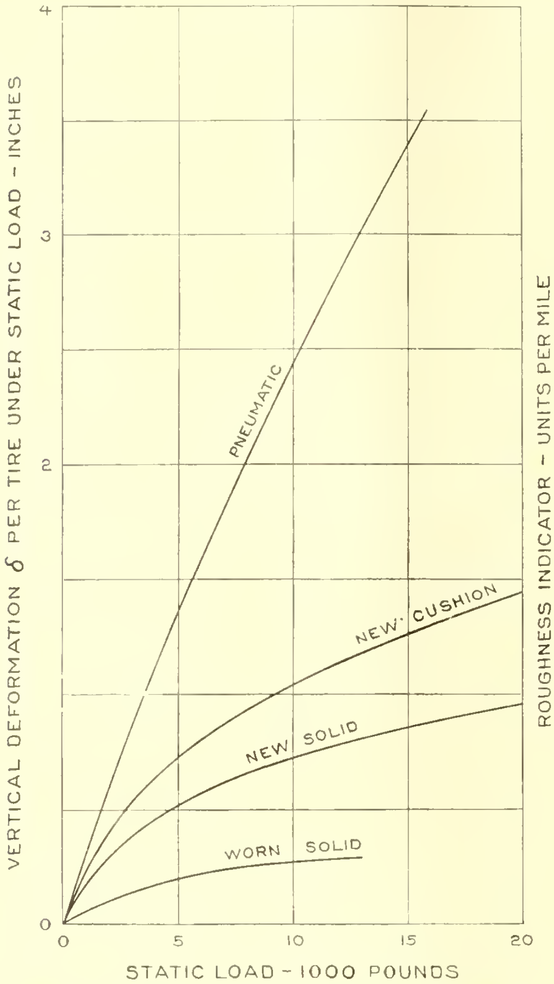


FIGURE 4.—AVERAGE STATIC LOAD; DEFLECTION CURVES OF THE TIRE TYPES USED

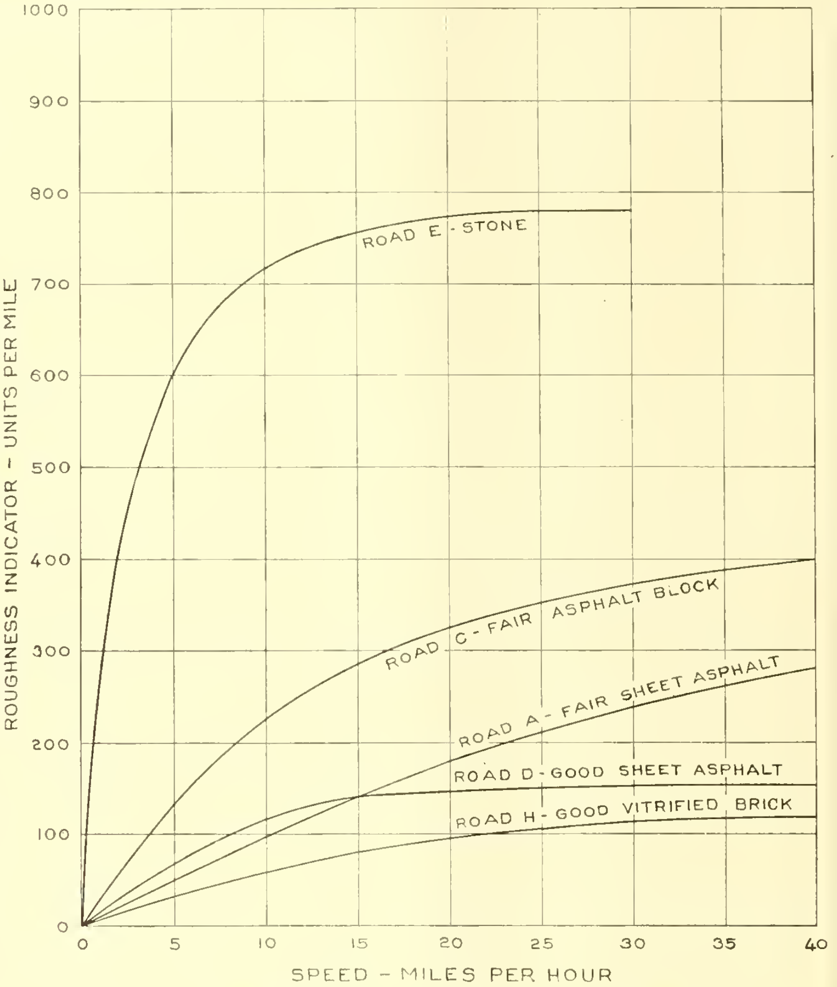


FIGURE 5.—CALIBRATION OF ROUGHNESS OF TEST SECTIONS WITH RELATIVE ROUGHNESS INDICATOR

impact tests. The average speed for each run was computed from a stop-watch measurement of the elapsed time during which the truck was passing over the measured test section.

The five greatest accelerometer deflections were then measured and the average deflection (of these five) obtained for each run. These data were listed as illustrated by Table 7. In the field, a rough check was obtained at this stage by plotting average accelerometer deflection against truck speed and checking any doubtful points before changing to the next test condition.

The average accelerometer deflections and truck speed were next listed on computation sheets and total forces computed, as illustrated by Table 8, according to the following formula:

$$F = S + mcr,$$

where

F = impact force for right rear wheel,
S = static total weight per rear wheel,
m = mass of unsprung weight per rear wheel,
c = accelerometer rate for the given conditions,
r = accelerometer deflection.

A preliminary work sheet was made on which impact force was plotted against speed and curves drawn through these points. Figure 7 is a typical example of a work sheet and is based on the data given in Table 8. From the smooth curves drawn on the work sheets, readings were taken at regular intervals for plotting on the isogram sheets. In Figures 8 to 12 the small cross symbols represent points so obtained and the corresponding forces in thousand-pound units are indicated by the adjacent numerals.

The portion of Figure 10 representing new solid tires is directly comparable with the typical work data above described. The construction of the actual isodynamic curves in 5,000-pound intervals was then made by a process (similar to the construction of contour maps from plotted survey computations) of obtaining points on the desired curves by interpolation and the drawing of smooth curves through the points so obtained. In Figures 8 to 12 the small filled circles indicate the interpolated points.

ISOGRAMS USED TO PRESENT DATA

A separate isogram was made for each test road according to the above method. As in the original isogram for the test road at Arlington Farm, the coordinates are wheel-load and truck speed, separate graphs being made for the four types of tire. The curves are lines of equal impact reaction and are drawn in increments of 5,000-pound reaction. It is again mentioned that the impact reaction is the average value of the greatest impact forces reasonably expected to occur as frequently as one hundred times per mile.

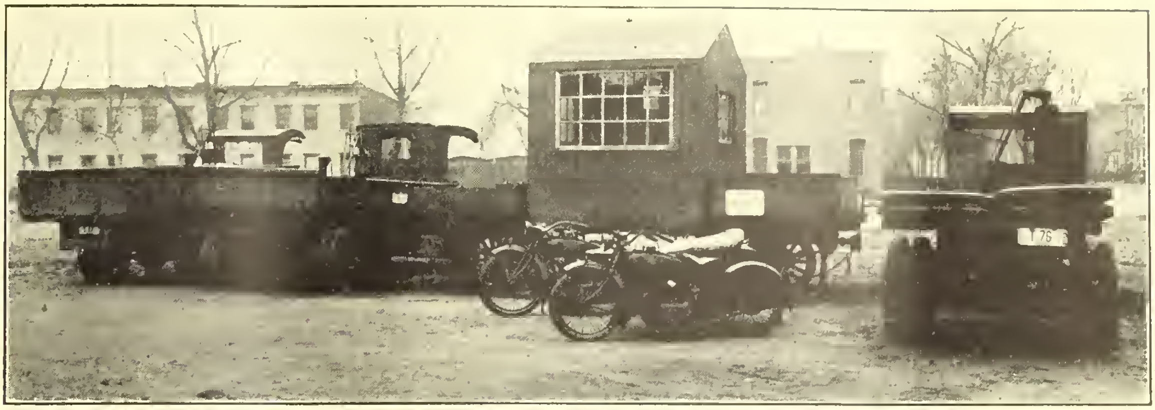


Figure 6.—Equipment Used in Tests. On the Left are the 2-Ton and 5-Ton Test Trucks. In the Center is the Trailer Equipped With a Field Office and Two Loading and Storage Platforms. On the Right is the Service Truck on Which is Mounted a Chain Hoist for Changing Tires. The Motor Cycles Were Used by Local Traffic Officers

In making the selection of road surfaces for these tests it was necessary to give consideration to three conditions. First, they should include the extremes of surface condition from very smooth to very rough; second, they should furnish a sufficient number of types so that the data would cover the range between these extremes as well as possible; and, third, the pavement selected should be of such character that they would not change in condition during the tests. It would have been very difficult to fulfill all these conditions perfectly. It is believed, however, that the sections selected represent a very wide range in roughness and that all of them maintained their characteristics during the tests, but with the pavements which were available in the vicinity of Washington, it was not possible to cover the range in roughness in as uniform a manner as might be desired. Referring to Figure 5, it will be noted that there is a gap between the curves for Roads C and E. However, it is thought that the majority of pavements carrying any considerable amount of traffic will be found to have surfaces as smooth or smoother than Road C, so that the deficiency just mentioned is not so important as might appear from a cursory inspection of the surface roughness data contained in this figure.

Table 7.—*Typical field data sheet from tests of fair sheet asphalt using new solid tires*

2,500-POUND WHEEL LOAD

Speed, m. p. h	27.2	23.1	22.0	19.1	16.7	13.0	10.8	8.1	5.3	16.4	20.0	17.0	18.0	23.7
Accelerometer deflections, inches	0.205 .177 .167 .162 .158	0.160 .140 .105 .098 .088	0.152 .150 .105 .096 .096	0.180 .126 .110 .102 .092	0.066 .055 .052 .050 .049	0.047 .035 .035 .033 .032	0.056 .032 .030 .023 .023	0.016 .014 .014 .013 .013	0.011 .003 .003 .003 .002	0.075 .063 .062 .050 .050	0.174 .163 .092 .086 .085	0.096 .072 .063 .063 .058	0.127 .080 .078 .076 .075	0.162 .137 .132 .110 .098
Average deflection, inches	.174	.118	.120	.122	.054	.036	.033	.014	.004	.060	.120	.070	.087	.128

5,000-POUND WHEEL LOAD

Speed, m. p. h	5.3	7.3	10.4	13.4	19.2	14.6	22.5	25.0						
Accelerometer deflections, inches	0.002 .002 .002 .000 .000	0.011 .008 .008 .005 .004	0.023 .022 .018 .017 .016	0.058 .035 .033 .029 .028	0.083 .078 .078 .076 .075	0.068 .058 .041 .040 .038	0.152 .137 .130 .125 .113	0.188 .173 .130 .116 .093						
Average deflection, inches	.001	.007	.019	.037	.078	.049	.131	.140						

7,500-POUND WHEEL LOAD

Speed, m. p. h	5.2	8.5	9.9	13.6	15.0	20.9	14.5							
Accerlerometer deflections, inches,	0.013 .010 .007 .006 .003	0.018 .015 .013 .012 .010	0.022 .018 .016 .015 .015	0.048 .029 .028 .028 .028	0.052 .036 .034 .033 .032	0.078 .078 .073 .064 .063	0.045 .037 .032 .028 .027							
Average, deflection, inches	.008	.014	.017	.032	.037	.071	.034							

10,000-POUND WHEEL LOAD

Speed, m. p. h	19.6	16.7	14.1	10.0	8.0	5.7								
Accelerometer deflections, inches	0.068 .052 .046 .045 .044	0.050 .046 .045 .033 .033	0.038 .033 .032 .030 .026	0.020 .013 .013 .013 .013	0.014 .013 .008 .008 .007	0.008 .006 .006 .006 .005								
Average deflections, inches	.051	.041	.032	.014	.010	.006								

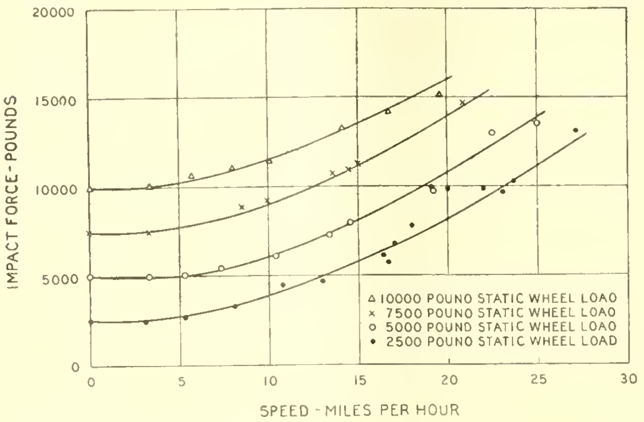


Figure 7.—Typical Work Sheet Showing Results of Tests on Road A Using Solid Tires

The isograms obtained for the five representative sections are shown in Figures 8 to 12, inclusive. In using them, the probable impact reaction (at the rate of 100 per mile of road) may be directly determined for any reasonable combination of wheel load and truck speed, whenever the road and tire conditions under consideration are identical with those of the isogram test conditions. If, however, the road condition or the tire condition (or both) should be different from those of the tests, then processes of single or double interpolation must be resorted to in applying the test data.

APPLICATION OF DATA BY PROCESS OF INTERPOLATION ILLUSTRATED

As an illustration, let us determine the impact reaction caused under the following assumed conditions.

Road: Intermediate in roughness between the sheet asphalt and asphalt block surfaces represented by Roads A and C. Let is be designated as Road X.

Tire: Intermediate in cushioning quality between new solid and worn solid equipment.

Load: 7,000 pounds per wheel.

Speed: 17 miles per hour.

So far as the assumptions as to load and speed are concerned, the reactions are read directly from the curves.

In determining the impact force for a condition which requires interpolation between two adjacent curves, it is recommended that interpolation be made through the point in question along a line approximately normal to the curves as shown on the graph for new solid tires in Figure 10. The curves were originally located by interpolations in both of the coordinate directions and the slope of the "nearest normal" determines the proportionate weights given to the load and speed influences.

In order to make the method clear, it will be illustrated by the following examples: As a simple case let it be assumed that it is desired to determine the probable approximate impact reaction for a truck equipped with new solid tires, having a rear wheel load of 7,000 pounds, travelling at 17 miles per hour over a road whose roughness is that of Road A used in

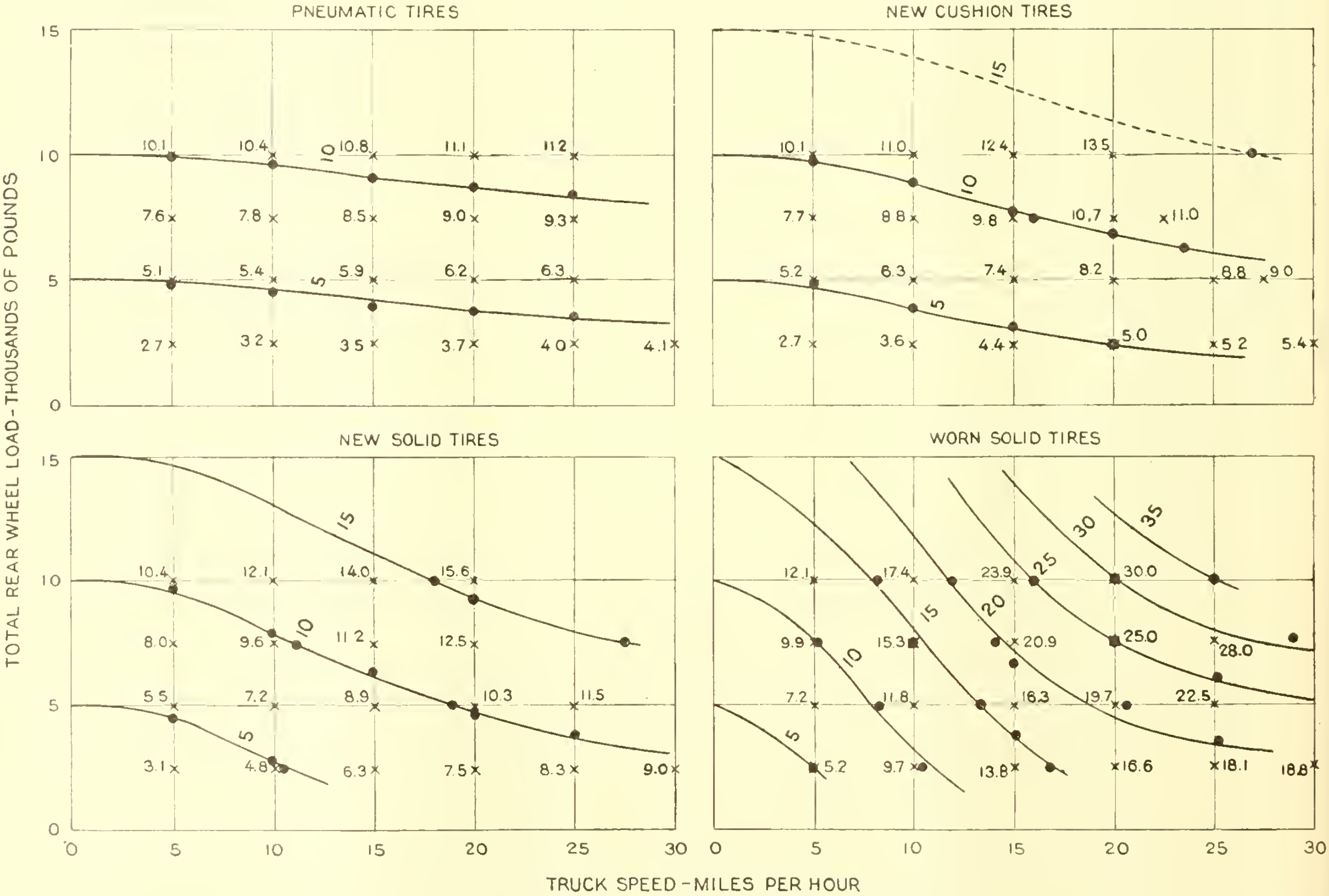


Figure 8.—Isodynamic Curves for Good Vitrified Brick Highway (Road H) for Determination of Average Total Vertical Reactions in Thousands of Pounds Occurring at the Rate of 100 per Mile

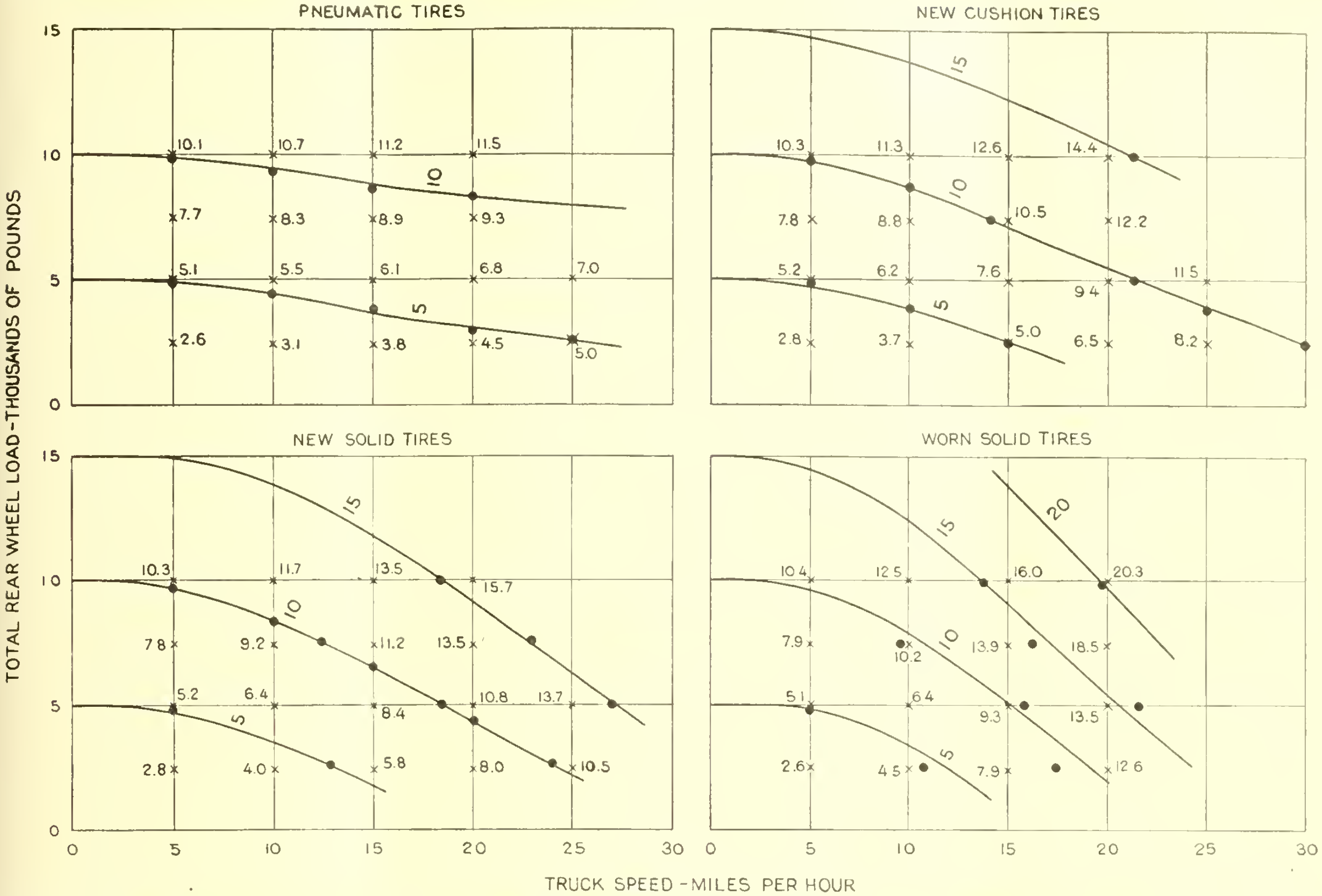


FIGURE 9.—ISODYNAMIC CURVES FOR GOOD SHEET ASPHALT HIGHWAY (ROAD D) FOR DETERMINATION OF AVERAGE TOTAL VERTICAL REACTIONS IN THOUSANDS OF POUNDS OCCURRING AT THE RATE OF 100 PER MILE

the test. Referring to Figure 10, and using the speed (17) and the wheel load (7,000) as coordinates, a point is located which lies between the impact reaction curves for 10,000 and 15,000 pounds. If a line is erected through this point approximately normal to the curves, as explained above, and an interpolation is made between the curves along this line, it will be found that the impact reaction indicated for the assumed conditions is about 11,500 pounds.

As a somewhat more difficult example let it be assumed that the roughness of Road X has been determined at the speed assumed in our example (17 miles per hour) at 250 units per mile. The simplest reasonably close interpolation between the two isogram road conditions is according to a straight line relation. At 17 miles per hour the roughness of Roads A and C are 158 and 302 units per mile, respectively. Reactions for Road X will be in excess of those for the smoother road by the proportional amount (indicated by the ratio of the road roughness differences) of the difference in reaction for Roads A and C. At the assumed speed, Road X is 92 units and Road C is 144 units rougher than Road A. Therefore, reactions for Road X are in excess of those for Road A by 92/144, or 64 per cent of the difference between the reactions for Roads A and C. The reactions are listed below.

	Road A	Road C	Difference	Correction	Road X
New solid tires	11,500	14,200	2,700	1,700	13,200
Worn solid tires	15,900	25,000	9,100	5,800	21,700

TABLE 8.—*Typical computation sheet showing force computations from average accelerometer deflections given in Table 7*

Instrument: 2a-2-L $m=23.0$ $c=2,655$ $mc=61,060$						Instrument: 3a-D-L $m=38.0$ $c=2,654$ $mc=100,850$					
Wheel load, 2,500 pounds			Wheel load, 5,000 pounds			Wheel load, 7,500 pounds			Wheel load, 10,000 pounds		
Deflection	Force	Speed	Deflection	Force	Speed	Deflection	Force	Speed	Deflection	Force	Speed
Inches	*Lbs.*	*M.p.h.*	*Inches*	*Lbs.*	*M.p.h.*	*Inches*	*Lbs.*	*M.p.h.*	*Inches*	*Lbs.*	*M.p.h.*
0.004	2,740	5.3	0.001	5,060	5.3	0.008	8,310	5.2	0.006	10,610	5.7
.014	3,350	8.1	.007	5,430	7.3	.014	8,910	8.5	.010	11,010	8.0
.033	4,510	10.8	.019	6,160	10.4	.017	9,210	9.9	.014	11,410	10.0
.036	4,700	13.0	.037	7,260	13.4	.032	10,730	13.6	.032	13,230	14.1
.060	6,160	16.4	.049	7,990	14.6	.034	10,930	14.5	.041	14,130	16.7
.054	5,800	16.7	.078	9,760	19.2	.037	11,230	15.0	.051	15,140	19.6
.070	6,770	17.0	.131	13,000	22.5	.071	14,660	20.9	------	------	------
.087	7,810	18.0	.140	13,550	25.0	------	------	------	------	------	------
.122	9,950	19.1	------	------	------	------	------	------	------	------	------
.120	9,830	20.0	------	------	------	------	------	------	------	------	------
.120	9,830	22.0	------	------	------	------	------	------	------	------	------
.118	9,710	23.1	------	------	------	------	------	------	------	------	------
.128	10,320	23.7	------	------	------	------	------	------	------	------	------
.174	13,120	27.2	------	------	------	------	------	------	------	------	------

IMPACT OF PARTIALLY WORN SOLID TIRES

Reactions for tire equipments intermediate between the new solid and worn solid types may be computed by adding to the reaction for new solid tires a certain proportional amount of the difference between the reactions for new solid and worn solid tires. We are able to obtain a rational factor for such use by

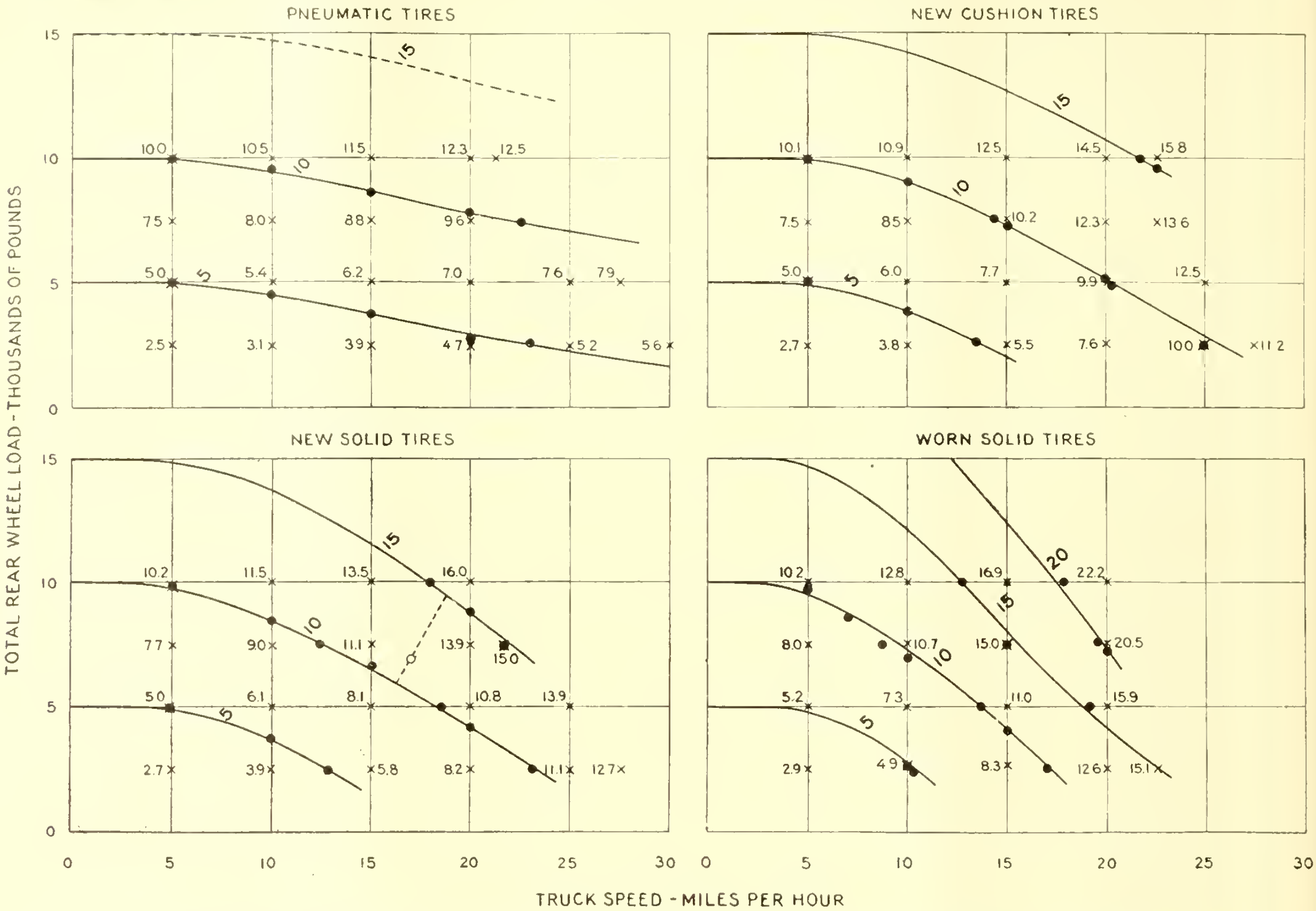


Figure 10.—Isodynamic Curves for Fair Sheet Asphalt Highway (Road A) for Determination of Average Total Vertical Reactions in Thousands of Pounds Occurring at the Rate of 100 per Mile

adapting data obtained on the artificially obstructed road in the separate research concerning the influence of tire height on impact reaction which is reported in this issue of Public Roads. This adaptation was necessary because the isogram tests involve a wide range in road roughness conditions and only two specific solid tire conditions. The influence of tire height on the proportional part of the difference between the reactions for new and worn solid tires as used in these isogram tests (which is the correction to be added to the reaction for new solid tires in order to arrive at the reaction for partially worn solid tires) has been computed from the data on cut-down tires presented in the other report and is shown in Figure 13, the limits in tire wear corresponding to the average heights of the two solid tire types used in the isogram tests. As the worn solid tires used in the tests on which the isogram curves are based had a loss averaging 68 per cent of the visible rubber on the new solid tires, the limits of loss in visible rubber in Figure 13 are 0 and 68 per cent and the corresponding proportional corrections are 0 and 100 per cent of the difference in reactions for the two types.

We are now in a position to compute the reaction on Road X, at the assumed load and speed, for a given condition of partially worn solid tire. If the height of visible rubber of this tire had been orginally, say, 2.10 inches, and in its partially worn condition was, say, 1.60 inches, then the loss in visible rubber would be 0.50 inch or 24 per cent of the original height. Referring to Figure 13 we obtain the factor 0.22 by which the difference between the reaction for new and worn solid tires is to be multiplied. This product is the correction to be added to the reaction for new solid tires to obtain the reaction for the partially worn tire. This computation is tabulated as follows:

	New solid	Worn solid	Difference	Correction	Partially worn
Road X	13,200	21,700	8,500	1,900	15,100

The reaction, then, for the conditions we have assumed for the four major variables is in round numbers 15,000 pounds, and should reasonably be expected to occur at the rate of one hundred times per mile or once in every 50 feet

In the report on tests on cut-down tires, it is indicated that for a given height of visible rubber there is substantially no difference in impact reactions for cushion tires and replacement sizes of solid tires. It is, therefore, suggested as reasonable that cushion tires be regarded as solid tires after the visible rubber has been worn to the height corresponding to that of the replacement regular solid tire. In general, this wear would be about 1 inch.

It is not essential to consider worn solid tires with less visible rubber than those used in the tests covered by this report because such tires are generally unfit for further service. Usually they are neither true nor round, have considerable variations in cross-sectional rubber, and have undergone aging and hardening.

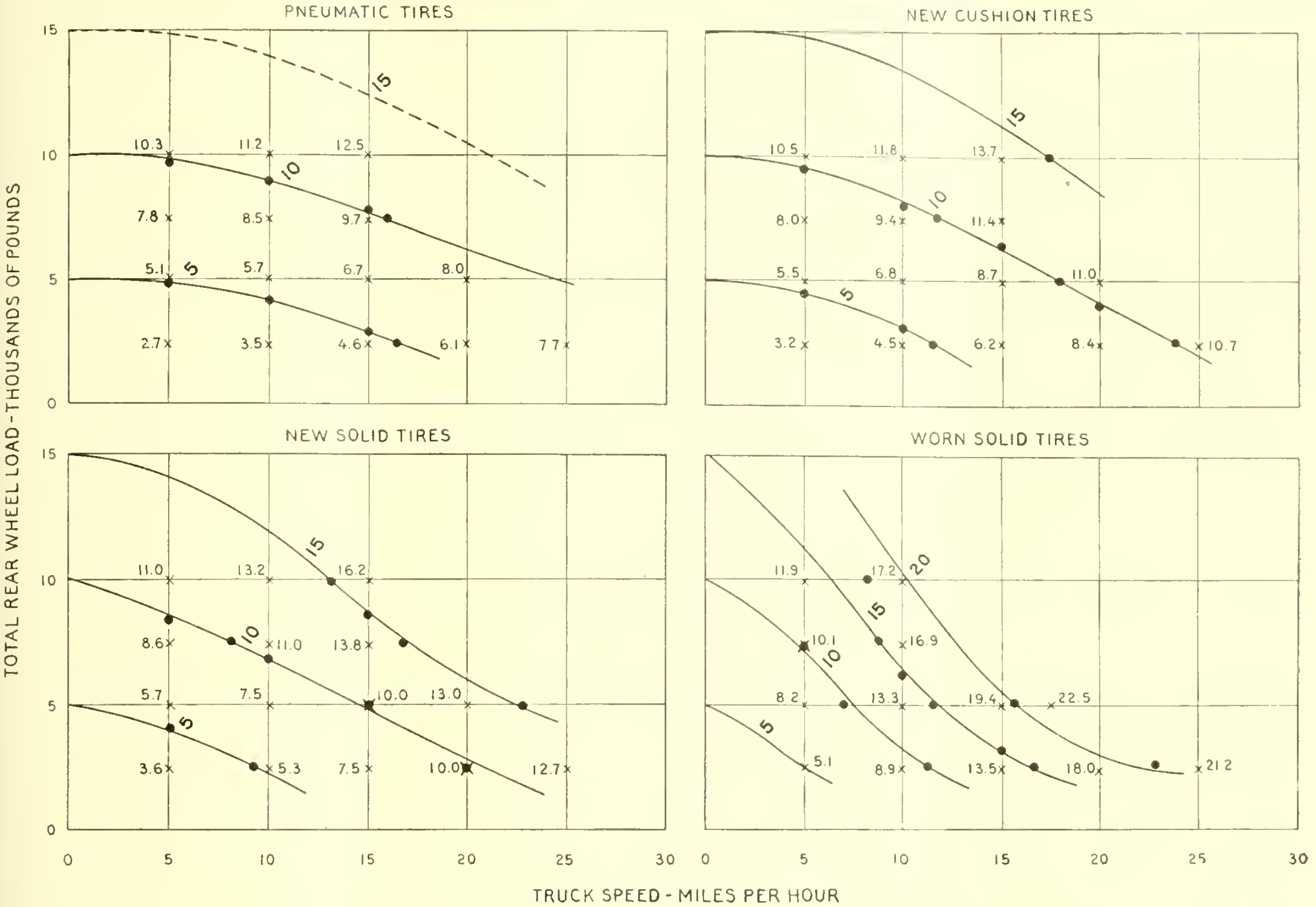


FIGURE 11.—ISODYNAMIC CURVES FOR FAIR ASPHALT BLOCK HIGHWAY (ROAD C) FOR DETERMINATION OF AVERAGE TOTAL VERTICAL REACTIONS IN THOUSANDS OF POUNDS OCCURRING AT THE RATE OF 100 PER MILE

The worn tires, it has been pointed out, represent a loss in visible rubber amounting to about 68 per cent of that of new regular solid tires, the average residual height of visible rubber being about 0.68 inch.

As the worn tires of these tests were obtained by cutting new tires down to the desired height, they were of live rubber, true and round. For this reason, the results obtained with tires designated as worn are considered to represent conservatively the reactions for this general class.

In determining the factor to be used for any partially worn solid tire, according to the relation shown in Figure 13 and under present conditions of operating practice, it appears reasonable to assume, in general, that the height of visible rubber of new solid tires is the average height of the new regular solid tires used in these tests. This height is 2.2 inches.

ISOGRAM DATA SUGGEST BASIS FOR REGULATING VEHICLE SPEED ACCORDING TO TIRE EQUIPMENT

In the effort to control motor truck traffic on the highways so that its destructive impact effect will be minimized, a number of States have enacted laws which limit the speed of motor vehicles according to the tire equipment. Up to the present, however, there has been little or no information available as a rational basis for such laws. In the study of the data obtained in the present series of tests it was found possible to establish speed differentials which, when applied to the various types of tire used, would yield impact reactions of approximately the same magnitude and thus, for the first time, at least tentatively relate the variables of vehicle speed and tire type.

The data are by no means exhaustive for a study of this kind since the establishment of differential speed relations was not one of the foreseen objects of the tests. The values given in the following discussion and in Table 9 should not be considered as conclusively established, but they may be regarded as tentative differentials for relating in a general way the main tire classifications.

For the purposes of this comparison, the speeds at which each type of tire would cause the same impact reaction as that caused by the pneumatic tire at a maximum speed were tabulated for all the roads and for wheel loads ranging from 3,000 to 10,000 pounds, by 1,000-pound increments. The average speeds, when expressed as a percentage of that for pneumatic tires, appear in round numbers, as follows:

	Per cent
Pneumatic tires	100
New cushion tires	55
New solid tires	45
Worn solid tires	25

The above tabulation indicates, as a general average for the conditions of these tests, that the impact reactions with pneumatic tires at a maximum speed will be reproduced by other tire equipments at the respective percentages of the speed at which the pneumatic tire is operated. The relations for a convenient range of the variables involved have been listed in Table 9. For general use the classifications of tires should be as few as is practicable. The suggested

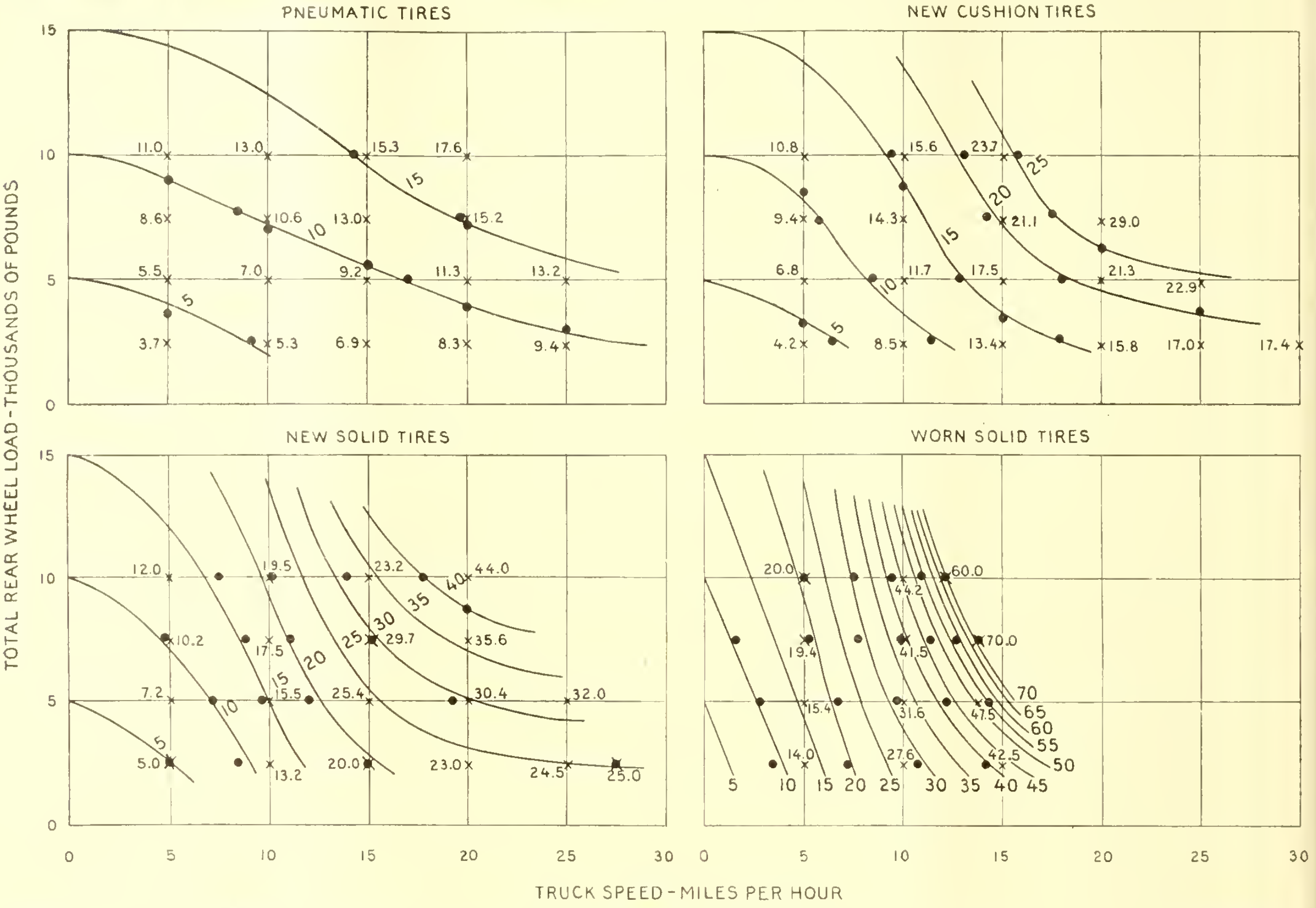


Figure 12.—Isodynamic Curves for Stone Highway (Road E) for Determination of Average Total Vertical Reactions in Thousands of Pounds Occurring at the Rate of 100 per Mile

percentages in Table 9 are not in every case the actual ratios of the computed averages because allowances have been made in order to approach more nearly average conditions of tires in service.

Table 9.—*Speeds at which approximately equal impact reactions would be produced by various types of tire equipment, as indicated by these tests*

Tire equipment	Relative speed						
	Per cent	*m.p.h.*	*m.p.h.*	*m.p.h.*	*m.p.h.*	*m.p.h.*	*m.p.h.*
Pneumatic	100	40	35	30	25	20	15
New cushion	55	22	19	16	14	11	8
New solid	40	16	14	12	10	8	6
Worn solid	20	8	7	6	5	4	3

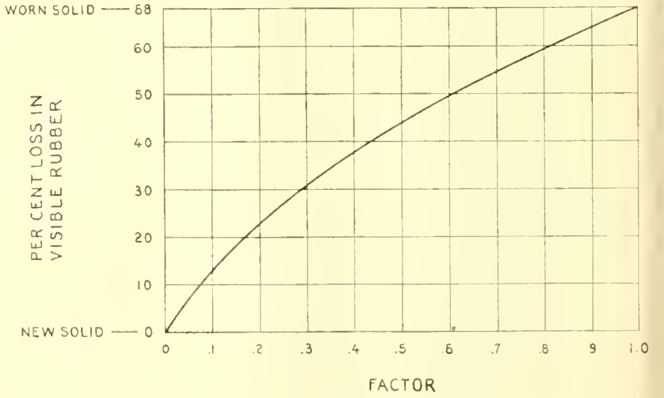


Figure 13.—Diagram Showing Factor for Use in Evaluating Partially Worn Solid Tires Intermediate Between the New Solid and Worn Solid Isogram Types

It will be noted that it is suggested that cushion tires be permitted to operate at 55 per cent of the speed allowed for pneumatic tires, and this is the actual average value found in the analysis of the test data. The tests reported in the article on pages 133 to 138 have indicated two conditions which were considered in arriving at this figure in which no allowance is made for slight wear found on nearly all tires in service. First, the effect of the loss of a unit height of visible rubber on the impact reaction increases as the amount of rubber worn off increases. The loss of the first inch in visible rubber height is not nearly so serious as are subsequent losses of 1 inch in height. Second, cushion tires and solid tires have practically the same cushioning properties when the load carrying capacities and the height of visible rubber are approximately equal. In this connection it has been suggested earlier in this report that the cushion tire designation be discontinued after such tires have been reduced to the height of the replacement sizes of regular solid tires. If these two indications are accepted as sound, they lead to the conclusion that cushion tires, so long as they are so classified, should be allowed a speed differential substantially the same as that indicated by the data from these tests, which were conducted with new cushion tires.

The tests indicate that new solid tires should be allowed to operate at a speed 45 per cent as great as that for pneumatic tires. As the average solid tire in service has undergone some wear, it is suggested that

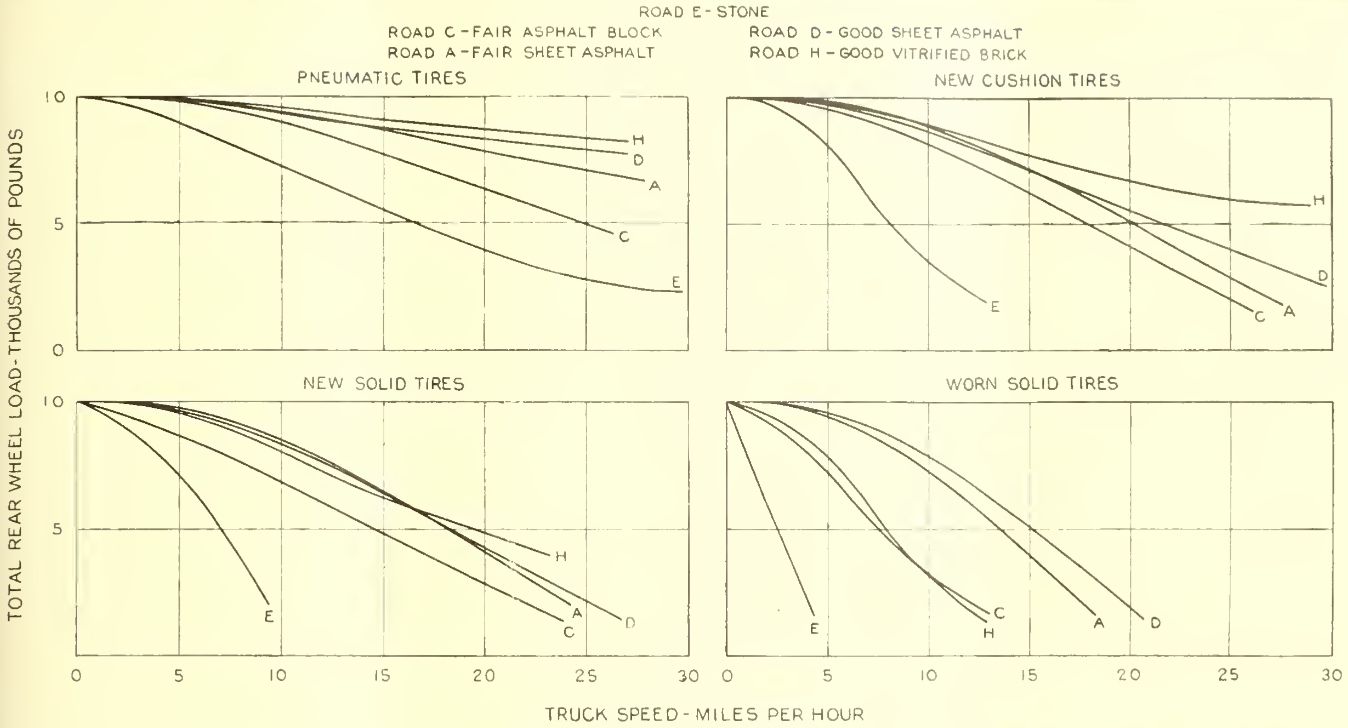


Figure 14.—Isodynamic Curves of Various Highways Based on 10,000-pound Total Vertical Reaction Occurring at the Rate of 100 per Mile

the speed for this class be lowered to 40 per cent of that permitted with pneumatic tires. This should give a differential better suited to the average condition of this class of tire.

Tires which have been worn down in service to the degree represented by the worn tires in the tests, in addition to having been subjected to aging and hardening influences are, as a rule, unevenly worn and are neither true nor round. Without question such service tires would cause appreciably greater reactions than those obtained with the test tires. As it would be practically impossible to select service-worn tires as being average or representative specimens, it was decided to cut down new tires to the height of rubber desired for the tests. It was realized that such test tires represented the best possible condition for worn tires and because of this the data are believed to be very conservative, and further, that any speed differential selected on the basis of these data should be considerably modified before it could properly be applied to the worn-tire class as a whole. For this reason the suggested value for this type is four-fifths of the actual value obtained from the isogram data.

OTHER ISOGRAM ARRANGEMENTS ILLUSTRATED

The isograms for the five road surfaces may be rearranged or regrouped in a number of ways in order to bring out various relations between the variable factors involved. An illustration of this is given in Figure 14, which shows the influence of the several surfaces on the combinations of wheel load and truck speed which result in a certain (10,000 pounds) impact reaction, the panels being for the several tire equipments. Similar graphs might be made for a series of impact reactions, such as 20,000 pounds, 30,000 pounds, etc.

The curves may be regrouped to show the influence of tire equipment on static wheel load and truck speed combinations which result in given impact reactions on given road surfaces. Other comparisons may be found desirable for application of the data to individual problems.

CHECK TESTS

Referring to Figure 5 it is noted that Road H is apparently smoother than Road D. A comparison of Figure 8 with Figure 9 and an inspection of Figure 14 shows that the impact data corroborate this for pneumatic, new cushion, and new solid tire equipments. The impact data for worn solid tire equipment, however, indicate that Road H is distinctly rougher than Road D. The reason for this apparent discrepancy is probably in the length of the contact between the tire and the road. Road D, being sheet asphalt, presents a continuous surface to the tire. On the other hand, Road H, being vitrified brick, presents joints approximately 3⅜ inches apart. The average lengths of contact of the four types of tire used in these tests are shown for the four wheel loads in Table 10, and these values show that the length of contact of worn solid tires is too short to overlap more than two joints of the vitrified brick (spaced 3⅜ inches) with the result that as each joint is passed it is the only joint in contact with the wheel. On the other hand, the other types of truck tire and the balloon tires on the passenger automobile which carried the relative roughness indicator had sufficient length of contact so that at least two joints were always within the contact length with a consequent decrease in the influence of the individual joints.

Table 10.—*Average length of tire contact with road surface in inches for various static wheel loads*

Type of tire	2,500 pounds	5,000 pounds	7,500 pounds	10,000 pounds
Dual pneumatic	7¼	9	10½	11¾
Dual new cushion	7¼	9	10	11
Dual new solid	6½	7½	8¼	8¾
Dual worn solid	3½	4¼	5	5¼

Although this theory appeared to be reasonable, it was thought advisable to rerun the tests with worn tires on the vitrified brick section, first, to check the findings for this particular test condition, and, second, to see if the isograms could be reproduced with reasonable accuracy. The original field tests on this surface were made in the latter part of November, 1926. Reruns were made during the latter part of January, 1927. The isogram drawn from the data of this check test with worn tires agrees very closely with the corresponding original isogram at the heavier loads, although the curves are somewhat steeper at the lighter loads. The discrepancy is not great, however, and is attributed mainly to weather differences; possibly there was a slight change in road roughness or in the condition of the tires which had not been used for a considerable period prior to the rerun tests. The check isogram is shown as Figure 15.

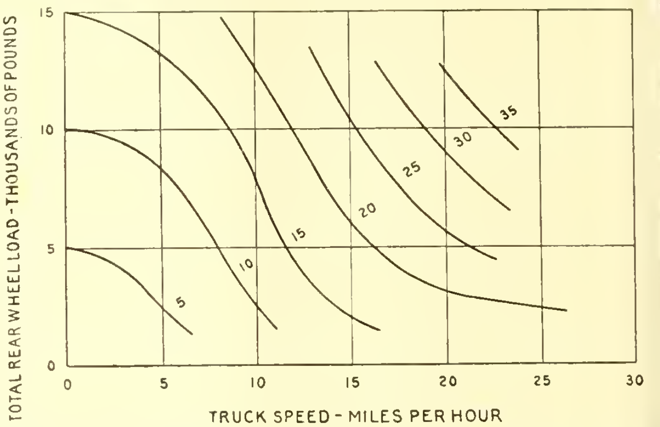


Figure 15.—Isodynamic Curves Resulting from Rerun Tests with Worn Solid Tires on Good Vitrified Brick Highway

POSSIBLE MATHEMATICAL TREATMENT DISCUSSED

Further studies have indicated that the data may be condensed by the use of generalized coordinates. If the generalizations be properly made and not too broad the accuracy will not be seriously affected. Further development of the data at the present time necessitates greater dependence on the indications of the roughness indicator than its status now warrants. Should this instrument eventually become standardized, it is not impossible that a formula may be developed for the computation of impact force for any combination of the four major variables, with an accuracy sufficient for most practical purposes.

(Continued from p. 138)

tire. This, again, should lead one to expect the difference between solid and cushion tires indicated in Figure 8.

As an example, consider the following cases for a 2-ton truck:

If the average height of visible rubber for a new solid tire were 2.1 inches, the corresponding average impact reaction would be about 340 per cent of the static load. If this tire were worn down to a height of visible rubber of 1 inch, it would have lost 1.1 inches of visible rubber, or about 52.5 per cent of what it originally had, and the average impact reaction would be about 220 per cent of that of the new tire. The reaction of the worn tire would thus amount to 2.20×340, or 748 per cent of the static load.

If the average height of visible rubber for a new cushion tire were 3.1 inches, the corresponding average impact reaction would be about 215 per cent of the static load. If this tire were worn down to the same height of visible rubber (1 inch) as the solid tire above mentioned, it would have lost 2.1 inches, or nearly 68 per cent, and the average impact reaction would be about 350 per cent of that of the new cushion tire. The reaction of this worn tire would thus amount to 3.50×215, or 753 per cent of the static load.

If the static load on the two types of tire were equal, the impact reactions for the tires worn to 1 inch of visible rubber would be approximately the same in pounds. If the tires were carrying their rated capacity loads, then the average reaction of the 4-inch solid tire worn to 1 inch of visible rubber would be approximately 7.48×2,000, or 14,960 pounds, and the average reaction of the 5-inch cushion tire worn to the same height of visible rubber would be approximately 7.53×1,700, or 12,800 pounds.

CORROBORATIVE TESTS

In a series of tests conducted in 1925 by Prof. J. T. Thompson at Johns Hopkins University[3] some relevant data were taken during the course of the investigations. Segments of a 3½-inch high-profile solid tire, each segment being cut to a different thickness, were fastened on the striking face of a trip hammer arranged to drop against a suitable anvil. It was found that the ratio of the impact force of the worn tire with respect to that of the new 3½-inch high-profile tire was 150 per cent when the visible rubber had been reduced by about 43 per cent and 240 per cent when the visible rubber had been reduced by about 71 per cent. The effect of the tire thickness on the impact force under these conditions was found to be generally comparable to the results found in the truck-impact tests.

TIRE-CUTTING MACHINE DESCRIBED

The tires were cut to the required thicknesses by a machine built for the purpose. The development of this machine was a by-product of the impact tests but it proved to be so satisfactory that a somewhat detailed description is included for the benefit of those who may desire to construct a similar device for truing tires which have become eccentrically worn. It consisted of the rear end of 2-ton truck securely mounted on a concrete platform, the left axle being locked and the differential driven by an electric motor through a belt and pulley reduction. The right wheel when mounted turned backward at the rate of about 90 revolutions per minute. A small lathe bed was arranged behind the right wheel so that the lathe carriage could carry cutting tools on the level of and parallel with the axle. Tires mounted on the wheel could be made to travel against cutting tools at the rate of about 15 feet per second. Figure 1 shows a general view of this apparatus with a tire with a completed cut mounted on the machine and a similar uncut tire shown at the right.

The technique in the use of this machine was developed as follows: A number of vertical cuts were made about one-half inch apart and to a depth within about one-sixteenth inch of the desired thickness. A horizontal cut was then made at the proper depth, carried beyond the first vertical cut, and the tool backed off. The first vertical cut was then completed with a manually held knife and the cut-off portion came off as a continuous band. The horizontal cut was then carried beyond the next vertical cut and the process continued until all the rubber to be removed had been taken off in a series of bands. The tire was kept wet to facilitate the cutting. By following this procedure, it was found that satisfactory finished surfaces which were quite true and smooth could be obtained.

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[3] See Public Roads, vol. 7, No. 5, July, 1926

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 8 7 OCTOBER, 1930

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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Vol. 11, No. 8 OCTOBER, 1930 G. P. St. CLAIR, Editor

TABLE OF CONTENTS

Page
General Reports Submitted to International Road Congress 153
International Road Congress Adopts General Conclusions 167
Bureau of Public Roads Enters Upon New Research Programs 172

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GENERAL REPORTS SUBMITTED TO INTERNATIONAL ROAD CONGRESS

THE SIXTH CONGRESS of the Permanent International Association of Road Congresses was convened in Washington on Monday, October 6, and closed on Saturday, October 11. Prior to the congress, 69 papers written by engineers in 20 different countries were published and distributed to the members. These papers deal with the following subjects, which were included in the agenda of the congress:

FIRST SECTION

Construction and Maintenance

First question. Results obtained by the use of:
  (*a*) Cement;
  (*b*) Bricks or other artificial paving.
  (Methods employed for road construction and maintenance in these materials.)

Second question. The most recent methods adopted for the use of tar, bitumen, and asphalt in road construction.

Third question. The construction of roads in new countries, such as colonies and undeveloped regions.

SECOND SECTION

Traffic and Administration

Fourth question. Ways and means of financing highways:
  (*a*) Road construction;
  (*b*) Maintenance.

Fifth question. Highway transport: Correlation and coordination with other methods of transport; adaptation to collective (organizations) and individual uses.

Sixth question. 1. Traffic regulation in large cities and their suburbs; traffic signals; design and layout of roads and adaptation to traffic requirements in built up areas.
  2. Parking and garaging of vehicles.

The reports on each question were reviewed by a general reporter (two for the first question), and these general reports were read at the meeting of the congress. It is a custom to appoint general reporters from the country in which the congress is held. For each question an American authority of recognized standing was selected to digest and summarize the reports submitted. The papers submitted by these general reporters are here reprinted in full.

RESULTS OBTAINED BY THE USE OF CEMENT

General report submitted by **FRANK T. SHEETS**, Chief Highway Engineer, State of Illinois

A thorough study of the papers on this subject submitted by reporters from many countries shows that the use of cement in both rural and urban road construction is steadily increasing on account of the inherent advantages of this material.

CEMENT DEFINED

In this discussion the term "cement" is assumed to mean the standard Portland cements and the closely related special or high early strength cements which have been developed in recent years. No attempt will be made to discuss so-called bituminous cements.

GENERAL USES OF CEMENT

Cement has been most generally used in road work for—

(1) The construction of cement concrete foundation or base courses on which were placed the many varieties of bituminous, brick, and paving-block wearing surfaces.

(2) The construction of cement concrete pavements which carry traffic on the cement concrete surface itself. Such pavements have utilized both plain and reinforced concrete and have been constructed in both single and double courses.

(3) The construction of cement-bound macadam surfaces; in which case three general methods have been used.

  (*a*) A dry mixture of cement and sand was rolled into a prepared stone surface and was then wetted and rolled until the resultant mortar flushed to the surface. After this set, the road was opened to traffic.

  (*b*) A layer of plastic mortar composed of cement and sand was sandwiched in between layers of macadam, and then the whole mass was rolled until the mortar worked into both courses of stone and sufficient mortar worked to the surface to produce a proper finish.

  (*c*) A thin grout composed of water, cement, and sand was poured into a previously prepared macadam surface, and the whole mass was then rolled and otherwise manipulated until a proper finish was secured.

CONCRETE ROADS SUITED TO HEAVY TRAFFIC

In general it may be said that cement concrete for foundation or base courses or for pavements which carry traffic directly on the concrete surface, as described in the foregoing paragraphs (1) and (2) has been used primarily on roads and streets which carry a large volume of vehicular traffic accompanied by frequent applications of heavy wheel loads.

CEMENT-BOUND MACADAM SUITABLE FOR MEDIUM OR LIGHT TRAFFIC

The cement-bound macadam types described in the foregoing paragraph (3) have been used with some success on roads and streets on which the density of traffic and weight of wheel loads were not inimical to the macadam type of construction. In such cases the addition of the cement binder seemed to give very good results, and greatly increased the traffic capacity over that which could have been carried successfully by ordinary macadam. The consensus of opinion seems to be that for light or medium traffic this type of construction, which costs less than regular concrete construction, has a place in the highway field.

STEEL-TIRED TRAFFIC REQUIRES SPECIAL DESIGNS

In some localities where steel-tired vehicles carrying heavy loads are frequent, cement concrete surfaces employing ordinary aggregates suffer seriously from abrasion. In such cases it has been necessary to supplement the structural strength or load-carrying capacity of the standard concrete pavement by using very hard coarse aggregate in the top or wearing surface of the pavement slab. This is the so-called 2-course method, and has given very good results at reasonable cost, provided the top course of concrete was applied before the lower course began to set. The 2-course concrete pavements seem to have met the abrasion problem satisfactorily and with much less first cost than that resulting from the construction of the bituminous surfaces or block surfaces on a concrete base.

SINGLE-COURSE CONCRETE PAVEMENTS SUITED TO RUBBER-TIRED TRAFFIC

Experience has shown that single-course cement concrete pavements which have been built carefully and with adequate thickness, but which have employed coarse aggregates ordinarily available rather than the excessively hard materials discussed in the preceding paragraph, withstand successfully almost unlimited amounts of rubber-tired motor traffic. This has been true even with frequent application of the heaviest wheel loads. The surface abrasion resulting from rubber-tired traffic is very small, and consequently in such a case the securing of adequate structural strength or load-carrying capacity at minimum cost is the primary function of a paving material. For this cement concrete is most suitable.

INHERENT ADVANTAGES OF CEMENT CONCRETE PAVEMENTS

It is obvious that the structural strength or load-carrying capacity of a pavement is measured by the strength of the supporting base. The various bituminous brick, or block surfaces with bituminous or sand fillers have little or no structural or flexural strength, and their primary function is to resist abrasion, reduce impact, seal the surface, or provide suitable appearance. Consequently, even when allowance is made for reduction in impact stresses due to such surfaces, the cement concrete base course under them must have approximately the same flexural strength, thickness, and cost as a cement concrete pavement which is to carry equal loads on the concrete surface itself. To state it simply, we must build practically the equivalent of a concrete road as a base for these other surfaces. This gives concrete pavements a great advantage in first cost. Of course where seasoned macadam bases are in place, this statement will not necessarily hold.

The selection of type of pavement to be used in a specific case can best be determined by the comparison of cost of service over a period of years. In such studies, first cost, interest on the investment, maintenance cost, and reasonable life of the pavements must be considered. Cement concrete, on account of its reasonable first cost with resultant low interest charge, its reasonable maintenance cost, and long life, shows up very favorably in such studies. This accounts for its increasing popularity and probably explains the fact that 59,000 miles of this pavement have been built in the United States of America up to the end of 1928, and 8,756 miles were built in that country in that year alone.

COMPETENT ENGINEERING SUPERVISION NECESSARY FOR GOOD RESULTS

Successful results from the use of cement concrete pavements or base courses can only be expected when all features of the work are performed under competent engineering supervision. This must include the preparation of the general plans showing modern grades, curves, drainage, and incidental structures, the rational design of the pavement slab to meet the conditions and to carry the loads which must be encountered, the preparation of rigid and detailed specifications to govern the work, and rigid inspection of the materials used and of the construction itself. This challenges the engineer to keep abreast of the times, to use scientific rather than rule-of-thumb methods, and shows the paramount necessity of a continuing and adequate program of research into the many unsolved problems yet confronting the engineering profession.

GENERAL SUMMARY OF PRESENT CEMENT CONCRETE PAVING PRACTICE

1. *Subgrade.*—The most important consideration in the preparation of subgrade is uniformity. Yielding areas should be replaced with soils of stable character. The subgrade should be well drained to eliminate surface water and that supplied from underground sources. The retention of smoothness of the finished pavement depends in large measure on care in preparing the subbase. The subgrade should be thoroughly wetted before concrete is placed, so as to avoid drawing moisture from the concrete during the setting process.

2. *Design of cross section (slab thickness).*—The minimum thickness of slab used has been approximately 6 inches. Theoretically, it should be of such thickness as will distribute the load from the surface to the foundation, so that the resultant pressures will not exceed the safe load for the foundation material. Earlier pavements have been uniform in thickness, or even thicker in the center than at the edge. More modern practice utilizes the extensive tests which have been made, which show definitely the need of the thickened edge section. Exposed corners and edges are obviously weaker than the interior of a slab having uniform depth. It is all but universally recognized that the thickness of the mid-portion of a concrete pavement need be only about seven-tenths of that near the edge, in order that the flexural strength throughout may be approximately equal. It is generally believed that if the edge thickness is proportioned in accordance with the formula $t=\sqrt{\frac{3W}{0.5M}}$ reasonable safety of design will result. In this formula, t = edge thickness, W = maximum expected wheel load, and M = modulus of rupture of concrete.

3. *Reinforcement and joints.*—The design of reinforcement should be in accordance with the weight of traffic. On stable foundations, pavements need but slight reinforcement; but on the other hand, where unstable foundations are encountered, steel reinforcement properly placed gives the slab increased resistance

to flexure. Reinforcement is effective also in preventing large shrinkage cracks.

The principal function of reinforcing steel as commonly used in pavements is to cause a dowel or shear action between edges at joints or cracks across which the steel passes.

Longitudinal joints in pavements have met with much favor, as they eliminate irregular longitudinal cracking. Practically no longitudinal cracking ever occurs in pavement slabs 10 feet or less in width. Best practice seems to call for such joints at about this interval. These joints are commonly doweled by means of the form of the concrete and by short transverse bars placed at frequent intervals.

Transverse expansion joints are commonly employed in concrete pavements, but opinions differ as to their location. The spacing varies from 20 feet in plain concrete on up. A large mileage of successful pavements has been built without such joints. Joints constructed on an angle to center line of the pavement in order to reduce vibration of traffic are not justified because of construction difficulties and the failure of the slab at the acute angles. The breaking of joints is not satisfactory, as cracks will continue across the adjoining slab. Pure bitumen or fibrous matter impregnated with bitumen are the materials usually employed in filling joints. Research should be directed toward finding a form of joint which will best transmit loading from one slab to another and a satisfactory form of joint material. Alternate bay construction has had some attention.

4. *Design of mixtures and materials.*—Advance in the art of concrete road construction has caused extensive research in materials and in the proportioning of aggregates. The scientific design of concrete mixes and the proportioning of aggregates by weight is gaining in favor. Standard specifications for aggregates can only be a form indicating certain points to be covered, as requirements must be adjusted to satisfy local conditions. It is universally agreed that only clean, hard, weather-resisting material, free from organic matter, should be used. Continued efforts are being directed toward the use of aggregates and cement in such proportions as will effect the greatest economy in design of the mixtures, strength, density, and cost, each being considered.

5. *Construction and equipment.*—Surface smoothness, correct depth of pavement, and the quality of the concrete are the principal items to be controlled in the construction of the pavement. Rigid specifications, thorough inspection, and penalties enforceable on contractors are necessary to get good results. Surfaces are commonly built so as not to vary more than one-fourth inch from a 10-foot straightedge. Beams and cylinders cast at the time the pavement is constructed are tested to determine flexural and compressive strength of concrete. Cores are drilled from the finished pavement to determine depth of pavement and also quality of the finished concrete.

The forms of curing commonly employed are:

(*a*) Ponding.
(*b*) Wetted earth, straw, or burlap.
(*c*) Surface application of calcium chloride.
(*d*) Surface application of sodium silicate.
(*e*) Surface application of bitumen.

Opinions vary as to the relative merits of these methods. In two or three different countries the surface application of a bituminous coat caused excessive cracking. Water curing is the accepted standard.

Most paving operations can easily be adapted to use of machines, and consequently paving equipment is constantly being improved to keep pace with modern construction practice.

6. *Maintenance.*—The increasing mileage of concrete roads has stressed the problem of maintenance. Cracks and joints are serviced at least once yearly, being filled with bituminous materials. Where pavement shows signs of disintegration it is covered with a mixture of bitumen and aggregate. General surface coats of bituminous materials, however, seem unnecessary and usually increase maintenance costs. If failure is of a critical nature, as in the case of sharp breaks, the area of pavement affected is replaced. High early strength cements are especially desirable for this type of patching work.

GENERAL STATEMENT OF CONCLUSIONS

1. Cement is becoming generally used as a paving material and has many inherent advantages.

2. It has been used successfully in the construction of cement concrete base courses for other surfaces, for cement concrete pavements, and for cement-bound macadam.

3. Cement concrete pavements and base courses are suited to heavy traffic.

4. Where a large volume of steel-tired traffic is encountered, 2-course concrete pavements, with the upper layer composed of very hard aggregates, have been necessary and successful. Other surfaces on concrete base courses have also satisfactorily met this condition.

5. Single course pavements have successfully carried maximum volumes of traffic and maximum wheel loads when the traffic was largely rubber tired.

6. Cement-bound macadam has been successful on roads carrying light traffic not inimical to the macadam type of construction.

7. Cement concrete pavements, when compared with cement concrete base courses surfaced with other materials, give equal load-carrying capacity or structural strength at less cost.

8. Competent engineering supervision of design, construction, and maintenance of cement concrete pavements is necessary to insure good results.

9. Subgrades must be uniform and stable.

10. Pavement slabs must be designated to carry expected loads. Edge thickening is necessary.

11. Longitudinal and transverse joints are commonly used and must be designed to meet traffic, subgrade and climatic conditions.

12. Scientific design of concrete mixes and weight proportioning of aggregates represent the most modern practice.

13. Construction operations are performed mostly by machinery, with resulting lower cost and better workmanship.

14. Thorough curing of concrete surfaces is essential.

15. Maintenance of concrete surfaces, properly constructed, is relatively simple and reasonable in cost.

RESULTS OBTAINED BY USE OF BRICKS OR OTHER ARTIFICIAL PAVING

General report submitted by P. J. FREEMAN, Chief Engineer, Bureau of Tests and Specifications, Department of Public Works, Allegheny County, Pittsburgh, Pennsylvania

From the information given by the reporters for various European countries, brick or other artificial pavements are not used to any great extent in Denmark, Italy, Irish Free State, Spain, Sweden, and Switzerland.

Brick is by far the most important artificial road material used in Holland, and the manufacture of brick has always been a very important industry. Of the national roads embracing in general the more important highways with a total length of 2,200 kilometers, more than 1,200 kilometers are made of brick, having a total surface area of approximately 4,500,000 square meters. In addition to the national roads a large part of the provincial roads are also made of brick. In municipalities, likewise, a brick surface has very frequently been used. About 52 per cent of the total surface of streets in Amsterdam are paved with brick and in The Hague 43 per cent of the streets are paved with brick. The annual production in Holland amounts, at present, to 500,000,000 bricks.

At the present time no uniform method of testing brick for all of Holland has been established. There are a number of tests prescribed by various authorities such as water absorption, compression, wear by means of a sand blast, drop test, and the ball compression test. Of all of these requirements the ball compression test appears to give the best indication as to the durability of the paving brick. The method is as follows: For the ball compression test six brick, of which one is broken in two, are laid with broken joints in three rows of two each in a shallow box lined with sand. The central brick is then loaded through a ball 19 millimeters in diameter and the applied load is gradually increased until rupture occurs. The mean load for 10 tests should be between 3,000 and 3,500 kilograms, with the minimum requirement of 3,000 kilograms for the lowest value permissible. The variations from the average value should not exceed 10 to 15 per cent.

In general, the depth of paving brick used in European countries has been less than that in the United States and the amount of vitrification has also been lower than in the United States.

In 1928 the Minister of Public Works of Poland sent experts throughout western Europe to study paving production with the idea of raising the standard of the industry in Poland to the highest technical level. Experience in Poland had shown that roads paved with brick having a good quality were durable under military traffic and with the recent development of high-class paving brick further extension of the use of that material for paving may be expected.

In Belgium, France, and Germany experience has shown that the durability of brick pavements is affected by the quality of the paving brick and more attention is being paid to the methods of making and burning paving brick.

Contrary to the practice in the United States, where paving brick is considered one of the most suitable materials for very heavy traffic, both as to number of vehicles and loading, it appears from reports that in continental Europe brick paving has been most generally used for light and medium traffic.

In many cases good European practice indicates that the paving brick should not be placed on a rigid base, particularly one made of concrete. Many of the brick pavements are laid directly on the subgrade or upon a heavy layer of granular material such as sand, which provides adequate drainage.

It has been found that unless the brick are of the highest quality they tend to pulverize under heavy traffic if laid on a solid base or foundation.

Under certain conditions it has been found unnecessary to fill between the brick with any material as the sweepings from the streets soon adequately fill the spaces between the brick. Pavements in the country are filled with sand or bituminous materials where traffic conditions make this necessary.

The countries of continental Europe are all developing paving brick of a higher quality than in the past and these countries are building experimental pavements under the most modern methods of design. This is also true of Great Britain, where the first experimental brick pavement was laid in 1922.

It would appear that the development of paving brick can be advanced if a uniform specification acceptable to the various countries could be developed. At the present time there does not appear to be any uniformity as to the physical requirements for paving brick used in various parts of the same country.

Through the efforts of the National Paving Brick Manufacturers' Association the rattler test for abrasion loss of paving brick has been made a standard by the American Society for Testing Materials and this test has done much to standardize the methods of production and the quality of paving brick in the United States. This test is used to a limited extent in European countries but it is not in general use.

An attempt will be made to summarize the general trend of methods for the construction of brick pavements in the various countries reporting:

It is agreed that proper drainage is a vital factor in the production of a durable pavement and that if the soil conditions are not favorable to proper drainage it will be necessary to provide artificial means for draining the subgrade.

It has been found in both Europe and the United States that in sections where drainage and climatic conditions are suitable it has been possible to construct satisfactory brick pavements on sand or well-drained earth subgrades. Where such foundations are inadequate to support the traffic the foundations are being constructed of brick, stone, blast-furnace slag or similar material, which is crushed, rolled into place and bound by means of fine materials, bituminous mixtures or Portland cement mortars.

For very heavy traffic plain cement concrete bases are commonly used, and in some cases the concrete is reinforced with steel bars.

There is no such thing as a standard adequate foundation for unit-type brick pavements. The facts in each instance must govern the design of the foundation and nothing should be overlooked to insure its being adequate. Best practice calls for a smooth base in which the variations shown on a 10-foot straight edge do not exceed one-fourth inch.

Fire clay and shale are used in the various countries for the manufacture of paving brick, and whereas the practice in European countries appears to be tending toward thicker brick, the reverse is true in the United States. Experience and tests in the United States have shown that brick having a depth of 2½ inches placed on a very smooth concrete base may be adequate for heavy traffic.

Having prepared an adequate foundation, a bedding course of fine material is spread over this foundation or base. This may be sand or slag screenings and for the best practice the thickness has been reduced to 1 inch or less.

After the bedding course has been screened to a uniform surface the paving brick are laid carefully with the best face up, with all lugs in the same direction, in parallel straight courses at right angles to the center line of the pavement. In some European countries the practice is to lay the brick to make an angle of 45° with the center line. The brick are then inspected and culled, after which they are rolled with a power tandem roller weighing from 3 to 5 tons. After final rolling the brick are again inspected and all unsatisfactory brick removed and replaced with perfect ones. A 10-foot straight edge is then used at various intervals across the roadway and any inequalities exceeding one-fourth inch are corrected and, if necessary, the entire surrounding surface again rerolled.

Portland cement grout filler is no longer in general use. Experience in France as well as in the United States indicates that it is not so satisfactory as bituminous filler. Asphalt is generally used for filler and the best modern practice is to apply the filler to the pavement by means of small-wheeled buggies equipped with rubber squeegees which will fill the joints without leaving a coating of asphalt on the surface of the brick thicker than one-thirty-second of an inch. For the materials used in the United States it is very necessary that the temperature of the asphalt filler as applied be not less less than 177° C. Specifications require the use of a large thermometer fixed in the kettle so that the temperature may be known at all times.

Immediately after filling the joints a thin coating of sand or granulated slag is spread over the surface of the brick and rolled to bed in the coating. Additional dressing may be added or the excess removed after the road is opened to traffic if the conditions warrant.

RUBBER PAVING

An artificial paving material which may have unusual future possibilities is rubber, which has been used in Great Britain since 1913. Several different types of paving blocks made from rubber are being tried. Some consist of solid rubber blocks 9 by 3 by 3½ inches deep, which are placed directly on a concrete foundation or other existing base. Other blocks of rubber have been used having dimensions of 9 by 4½ by 1½ inches mounted on a concrete base and held by chaplets of steel. Rubber pavements have been tried in the United States to a limited extent but a method of holding the rubber surface to the base has as yet to be perfected. Such pavements are not slippery and appear to have wearing qualities.

STONE BLOCK PAVING

Stone block pavements are considered natural pavements by the European countries reporting rather than artificial materials. In the United States block pavements have been constructed of granite, trap, sandstone, and limestone. The method of laying block stone pavements is practically the same as for paving brick. The joints are usually filled with a mastic consisting of approximately equal portions of sand and asphalt filler rather than pure bituminous filler as is the case with paving brick. In some cases cement grout filler consisting of equal parts of Portland cement and sand are used. Block stone pavements on adequate foundations are considered suitable for the heaviest type of traffic.

THE MOST RECENT METHODS ADOPTED FOR THE USE OF TAR, BITUMEN, AND ASPHALT IN ROAD CONSTRUCTION

General report submitted by ROY W. CRUM, Director, Highway Research Board, National Research Council, Washington, D. C.

Reports have been received from 13 countries telling of the successful use of tar, bitumen, and asphalt. Many combinations and methods of use are described. The rapid increase in the adoption of surface treatments to produce thin mat surfaces upon gravel, stone, and slag roads for light and medium traffic is probably the outstanding recent development. The methods described are well worth the careful study of engineers responsible for planning highway improvements.

Your reporter is greatly impressed by the many combinations of materials and construction methods used to stabilize earth, sand, gravel, stone, and slag surfaces for light traffic, and to form wearing carpets for heavy traffic. In Europe most attention has been directed toward the surfacing of existing macadam or similar roads, while in the United States there has been a great development in the initial construction of low and medium type roads. The developments have come about principally through road trials, although the influence of research, particularly into the characteristics of the bituminous materials, is notable. Intensive research methods, particularly into the combinations of bituminous binders with other materials and into economic factors, must now be invoked in order to develop further refinements in serviceability and economy.

The principal impressions received through the study of these excellent reports are as follows.

TERMINOLOGY

The nations do not yet agree upon the terms used in connection with the materials covered by the second question, the principal differences occurring because of the divergence in commonly accepted trade terms in Europe and America. New trade terms will be very slowly adopted. In the meanwhile a classification based upon the manner of commercial production of the various materials, giving the names by which they are known in the different countries, would be of great help in promoting understanding. The following cementitious materials are recognized in all countries: (*a*) Tar; (*b*) the solid or semisolid binders secured from natural deposits; (*c*) the solid, semisolid, and liquid binders produced by the distillation of petroleum; and (*d*) natural rock impregnated with bituminous binding media. The use of the term "bitumen" to describe that part of these various materials which is completely soluble in carbon disulphide appears to be generally acceptable. The difficulties arise when exact scientific definitions susceptible of a single meaning are attempted. Similar classification is also needed for the various basic road types which are now described by a great variety of trade names, many of which are not descriptive.

CONDITIONS OF USE

The reports describe the methods developed in the different countries, and give some information concerning traffic capacity, maintenance needed, and governing conditions necessary for success. Emulsions, owing to their adaptibility to use in wet weather, have had wide acceptance in Europe, while for use cold, cut-back products are more common in America. New types of equipment include a machine reported from Sweden for spraying sand and bitumen simultaneously from adjacent nozzles, so that each sand grain is coated; machines similar to those used on Portland cement concrete for finishing mixtures; attachments for pressure distributors for spraying asphalt and dilute soap solution to form an emulsion; and mechanical distributors for both coated and uncoated aggregates. Considerable attention is given to methods for overcoming slipperiness, particularly by rolling bitumen-coated stone chips into the surface. Applications of laboratory research into the constitution of bitumens are discussed in the Dutch report. The German report describes methods, which also parallel American practice, for design of mixture with especial reference to determination of correct binder content. The nonsuccess of surface treatments on certain roads in Spain carrying intense traffic of heavy carts is mentioned. European reports indicate increasing use of asphaltic bitumen, bituminous emulsions, and mixtures of tar with asphaltic bitumen.

NEEDS

It is apparent from the reports that tar, bitumen, and asphalt have been found satisfactory for use in road improvement throughout the world, and many of the factors upon which success depends are indicated. However, the thought occurs that further detailed discussion of the prevailing conditions, upon the evaluation of which depends the decision as to the particular type, design, and methods to use in specific cases, would be extremely valuable. Doubtless full understanding of the factors involved will depend somewhat upon further research into such economic features as transportation costs (roadway cost plus vehicle-operating costs) and relation between maintenance costs and intensity of traffic, and also upon research into the fundamental characteristics of combinations of bituminous binders and mineral aggregates.

CONCLUSIONS

I have the honor to recommend the following conclusions, which are based upon the impressions received through study of the national reports, for your consideration.

I. Tar, bitumen, and asphalt are suitable materials for use in the improvement of all classes of roads, subject to the limitations imposed by the characteristics of the material, the intensity of traffic, and local physical conditions. Certain governing conditions generally recognized are:

(*a*) The use of the correct grade and amount of bituminous binder for the particular type of work and aggregates to be used is essential.

(*b*) Provision for prompt maintenance is necessary. In general, successful results are attained when the roads can be kept in good condition at reasonable cost by maintenance of the surface without the need of extensive repairs to the underlying roadway.

(*c*) Proper proportions, thorough mixing, and thorough compacting are required in construction of premixed types of surface.

(*d*) Especial care is needed in the construction of bituminous macadam, since careless workmanship or improper methods may not be apparent immediately after construction.

(*e*) Attention should be given to the reduction of slipperiness.

II. The need for research into the fundamental factors involved in the use of tar, bitumen, and asphalt in road improvement and maintenance is recognized. The attention of the Members of the Congress is especially directed to needs as follows:

(*a*) Research into the constitutions and characteristics of bituminous materials, and their combinations with other materials.

(*b*) Research into factors affecting the serviceability and durability of roads, such as climate, subsoil, traffic density, and road design.

(*c*) Data on economic factors such as:

(1) Cost of transportation over various road types, including vehicle-operating costs and roadway costs.

(2) Relation between maintenance costs and volume of traffic for various road types.

III. In order that mutual understanding and helpfulness may be promoted, there is need for an international classification of materials, combinations of materials, construction methods, and road types which will correlate the terms in use throughout the world, both commercial and scientific.

THE CONSTRUCTION OF ROADS IN NEW COUNTRIES, SUCH AS COLONIES AND UNDEVELOPED REGIONS

General report submitted by E. W. JAMES, Chief, Division of Highway Transport, Bureau of Public Roads, United States Department of Agriculture

Twelve reports have been submitted on Question 3. The points on which agreement is most general are the need of complete studies of prevailing local conditions, the advantage of first constructing a limited system of main roads, the immediate expectation of motor traffic and the requirement of designs to accommodate it, the construction of temporary structures narrower than final plans will require, the possibility in some favorable localities of temporarily omitting surfacing, and the principle of salvaging construction.

It is interesting to note the emphasis in some reports on the military and political aspects of colonial road development, but one writer definitely states that military roads form a poor basis for later development.

The construction of roads in undeveloped regions of relatively old countries is described in several reports, and practically all agree that motor traffic must be expected and provided for in the premises.

Management and financing are touched upon by several reporters and the use of a general revenue is cited as required for initial work of a substantial character.

Standards suggested invariably reflect modern requirements for motor traffic roads. A 10-foot or 3-meter travel lane is preferred, but there is a marked difference of opinion as to whether first construction should consist of 2-lane roads or of 1-lane roads with passing places.

ALMQUIST reports for Sweden and describes the standards adopted by the Swedish Government for use in the land of the Laps in northwestern Sweden.

Widths of surfacing are 2.5 to 3 meters. Grades are limited to 10 per cent. A 100-meter radius is advocated for curves, but radii down to 10 meters are permitted when indicated by topography. Bridges are 2.5 meters wide. Passing places are provided along the roads at intervals of not more than 200 meters.

Roads built according to these standards are aided by financial grants amounting to 90 per cent of the cost by the National Government. The local communities pay the balance.

BOROWSKI advocates the completion of the highway net as earth roads for a country like Poland before hard surfacing any of the main roads.

He cites the difficulties experienced with poll-tax labor, and gives actual case figures indicating the economy of machine methods in the eastern districts of Poland.

He emphasizes the importance of a national plan, and gives the standards of design adopted by the First Polish Road Congress and notes the policies advocated by the second congress.

BRIX, reporting for Germany, refers to the political and military aspects of initial road construction in undeveloped areas. He points out that roads now built for the first time must be suitable for motor transport, but loads may be limited, and cites 2 to 2.5 metric tons as the maximum permissible.

Roads are preferred to railroads, but must correlate with them if they exist. Preliminary studies should include climate, materials, topography, soil, and economic needs of region. Main routes as a skeleton system are to be built first. In general 2-lane roads of 6-meter width should be designed. Surfacing may be narrow but must be placed to furnish all-weather service.

Narrow bridges are satisfactory temporarily, but clearances should be standard for single-lane traffic.

He describes the organization required and suggests financing by concessions, and believes, in general, that the same standards of design should be adopted as for road building in developed sections.

BRUCE and others, reporting on British and British colonial practice, point out especially the relevancy of the subject of undeveloped sections of old countries, and indicate the requirement in such cases of a thorough study and plan for correlating road construction with existing railroads. They emphasize the need of a high grade of personnel in such work, of the required permanence of alignment, and of sound, steady financing, suggesting the possibility of taxes on motor spirit and lubricating oils as a productive source for highway revenue.

The paper suggests also the value of airplane photographs in reconnaissance surveys, and in connection with location emphasizes the need of taking advantage of natural drainage.

Though temporary construction may be resorted to, the paper points the advisability of anticipating proper future bridge location and the elimination of railroad grade crossings. Although stating that the British Ministry of Transport has adopted the 10-foot unit traffic lane, the report indicates that widths of surface dictated by horse-drawn traffic are still used in several of the colonies.

The report advocates the abandonment of through intersections of minor roads with main thoroughfares, and proposes staggered intersections at such places. For intersections of two major roads, a square island is suggested for the purpose of developing rotating traffic. These proposals unquestionably reflect experience on a more highly developed road system in an older region, and from this point of view are interesting as details of location.

DE JONGE, reporting from Holland, describes the canal and highway transportation routes in the Province of Drenthe.

This was a typical undeveloped area in an exceedingly old country. Consisting of 266,000 hectares, the area had, in 1850, 82 kilometers of road; in 1928 it had 1,445 kilometers. The rights of way were 8 meters wide until 1917, and are now made 10 meters wide. Surfaces are 2.5 to 3 meters in width, and brick or stone setts are generally used.

Tables show the rapid recovery of agricultural land following road development, the rapid growth of population, and the increase of taxation.

DE SOLA, writing with a background of Cuban experience, emphasizes the value of surveys to locate suitable road-building materials in new countries in order to direct the intelligent selection of adequate types of surfacing and to determine economical methods of construction. He notes also the advantage of soil studies affecting the details of location. He calls attention to the requirement of developing the fullest possible use of natural drainage in fixing the location of new roads.

KURTZ reporting for Argentina furnishes a technical discussion of adequate mixtures for sand-clay road surfaces, basing his paper largely on Terzaghi's developments. He proposes a diagram to use the physical composition of mixtures, based on Terzaghi but using a metric scale, which he proposes to standardize so that comparisons of mixtures may be made by mere inspection after a physical analysis has been made and the results charted.

LAUZI describes the development of roads in Italian Colonies from the status of emergency construction for military purposes to the resurfacing with asphaltic tops of part of the permanent highway system of the Colonies. The use of macadam is emphasized and the development of local sources of stone. The road system is determined largely by local routes between existing sites, supplemented by militry routes required by the location of posts and camps.

PRINYAYOGAVIBULYA, reporting for Siam, describes the use of a cross section with reversed crown on new grade. This has been found useful in Siam, where heavy rains cause serious depreciation of new embankments. By this means erosion of the slopes is reduced; considerable rain water accumulates in the reversed crown, and expedites settlement of the embankments. Later, when the road is surfaced or further improved, a standard crown is constructed and the surfaces finished to usual cross section.

SKOGSTROM, of Finland, describes as an example of a development road the trunk line 229 kilometers long across Lapland, connecting the present road system of Finland with Petsamo Fjord on the Arctic coast.

VICAIRE cites especially development in Morocco where transitions are made directly to motor transport from pack trains. Development has followed closely the course in France. Main roads are very direct, with ample radius of curvature and low grades. Macadam 11 to 15 inches thick is used. Thirty-meter right of ways are required in open country, and not less than 16 meters even through settlements. Roadside planting is early promoted, and nurseries are already producing trees for this use.

Three trans-Saharan roads are projected, one of 2,100 and two of 1,500 kilometers. Two are open to traffic; one is under construction. Connecting laterals, 300 to 600 kilometers, are under construction.

In Tunis the completion of circuit routes for tourist purposes has been found expedient as first construction in undeveloped desert.

In Indo-China a principal artery 2,561 kilometers long has been built to penetrate a new area. Military trails, first used locally, show heavy loss in salvage value.

In West Africa, Togoland and Kamerun, the early use of bituminized surface is indicated as elsewhere in French colonies. The construction of single well-located cardinal roads for purposes of penetration, safety, trade, and administration appears necessary and adequate.

WHITE and others, reporting on the development of roads in well-settled agricultural areas in the United States, point out that any highway construction must advance in harmony with a possible, sound financial plan. Agricultural land can not carry required charges for extensive road system construction, and a supplementary source of revenue must be tapped. The anticipation of revenues of the future by resort to bond issues is considered sound.

Highway needs are increased by potential motor traffic in all regions, even the most sparsely settled areas. In a road system, third and second class, as well as first-class roads, must be made passable if they are to be of service. The very small amount of traffic indicates low-type improvement as adequate for the third-class group, but maintenance is needed in all cases to prevent deterioration.

Increase of traffic requires the heavier improvement, reclassification on a higher scale, and increased cost and maintenance charges. Types of surface treatment adequate for various densities of traffic are discussed, and standards of gradient and curvature are indicated.

A comprehensive highway system plan is the first essential. Right of ways 120 feet in width should be provided initially for main roads, 100 feet for secondary, and 60 feet for third-class roads. Directness of location without regard to property lines should characterize main roads. Only the third group of roads should regard ordinary land subdivisions. Alignment and profile should be made definite where possible; but surfacing should be determined by existing utilization.

HEWES and others, referring to road development in the sparsely settled western portion of the United States, consider method of financing of first importance, declare the sound principles of bonding an entire area (national or provincial) and of taxing developed areas to pay for first improvements needed in undeveloped areas. Maintenance and secondary construction should be carried by local administrative units. They cite development in western United States where automobile revenues have increased as a result of road improvements extended into undeveloped areas. Total motor revenues in 1915 of $2,800,000 have increased to $108,500,000 in same area in 1930.

The program of construction should be developed as early as practicable, should consider especially agricultural production, and recognize a forecast of automotive traffic. To avoid undesirable economies in construction, the program should not be forced in the early years. Final standards of grade and alignment should be introduced early. Bridges and surfaces may be made temporary on plans admitting salvage in later years. A method of oiling roads of light initial construction, composed of a variety of stone and gravel surfaces is described.

WAYS AND MEANS OF FINANCING HIGHWAYS: (A) ROAD CONSTRUCTION, (B) MAINTENANCE

General report submitted by ALVIN B. BARBER, Member American Society of Civil Engineers, Manager Transportation and Communication Department, Chamber of Commerce of the United States of America, Washington, D. C.

The subjects "Finance of the Construction and Upkeep of Roads" and "Provision of Revenues" were dealt with by the Third International Congress at London in 1913. The reports submitted in advance of the present congress contrast sharply with those submitted prior to the London Congress. The latter mainly considered highway systems as designed to carry the old forms of road transport and generally revealed no serious financial problems, whereas the predominant note in the present reports is the urgent need for extensive improvement, reconstruction, and new construction as well as better maintenance, to meet the demands of motor transportation.

The conclusions reached by the London Congress also presented a very different point of view from that reflected by the several reports submitted before the present congress. The discussion in the finance section at London, and the conclusions finally adopted, dealt chiefly with the financing of main routes of communications and those for long-distance traffic, whereas the present reports indicate that the main highway systems in most of the reporting countries are being fairly well provided for, but that financial problems of great magnitude are being encountered in the effort to accommodate secondary and local roads to meet the necessities of motor traffic. The motor vehicle is no longer a luxury or the exceptional case. It is essential in modern highway transport.

Where its use is well developed the motor vehicle is also able to pay its fair share of highway costs. In the sectional discussion at London a proposed declaration on the subject of a motor-vehicle fuel tax was rejected on the ground that conditions differed so much in different countries that no general statement on the subject was suitable for adoption by an international congress. The conclusions, however, suggested special taxes upon vehicles which, on account of weight or weight combined with speed or other exceptional circumstances, cause special damage to roads beyond the wear and tear of ordinary traffic. In contrast with that guarded expression the present reports show that practically all countries reporting now have motor-vehicle license taxes, while a majority have motor-fuel taxes and that they anticipate further great increases in highway revenues from these sources as the use of motor vehicles increases. While in some countries such revenues are not entirely devoted to highway purposes, the almost universal opinion is that they should be so applied.

The reports also generally recognize that the benefit to the general public resulting from highway improvement warrants finding a considerable proportion of needed funds from general taxation. This is particularly true as to local roads and urban streets, since user revenues are generally collected and expended by or under supervision of the national or other higher jurisdictions.

Assessment of abutting or other immediately benefited property continues but with a tendency toward restriction of its application to urban districts or, in other cases, to a portion of the increase in value resulting from the improvement.

The principle of aid to the lesser political units is being extensively applied either through outright subsidies to assist in developing roads of general use or through loans which must ultimately be repaid by the smaller units.

Another tendency shown is toward simpler classification of highways, the essential groups being two—first, those of general use serving a substantial proportion of through traffic moving within or through more than one local jurisdiction and, second, those of local use, including city streets. There is growing recognition, however, of the responsibility of the authorities in charge of main highway systems for the sections of such systems lying within municipalities.

Finally, following the principle briefly stated in the conclusions of the London Congress, there is general recognition of the investment character of improved highways which, making possible large increases in motor traffic, afford the basis for added revenues from user taxes which can be paid from the economies to highway users resulting from the improvements. Thus the issuance of bonds, under proper safeguards as to soundness of the highway program and its administration, length of the loan period and assurance of proper maintenance, are approved as a means of financing improvements which can profitably be expedited.

Reports from 11 countries have been received. With two exceptions they are from European countries with relatively dense population and closely knit systems. To bring out further views of less developed countries which are to be represented at the International Congress, cognizance has been taken, in preparing this report, of the conclusions on financing of highways adopted by the Pan American Highway Congress held at Rio de Janeiro August 16 to 28, 1929.

Following are certain outstanding features indicated by the reports from the several countries, in addition to which most of the reports give statistical data as to the amounts of the different classes of roads.

Denmark.—In 1926 motor taxes paid about 58 per cent and general taxes 42 per cent of the road expenditures of four typical counties in Denmark. The taxes on the user consisted of a fuel tax and graduated registration fees.

Most of the proceeds of the motor taxes, which are collected by the National Government, went to the counties and parishes, lesser shares being allocated to the cities and towns and a small amount (1 per cent) to a fund for special works. At least half the amounts distributed must be used for improvement, including bridge strengthening, provision of clear view at intersections, and protection of railway crossings. Motor taxes may be used to pay interest and amortization on loans for highway purposes.

The main (county) highways are nearly finished, but work on the "byroads" (parish highways) is just beginning.

France.—France has found difficulties in highway restoration since the World War. Maintenance of the national system, 7 per cent of the highways of all classes, is reported as backward. The report shows that the Departments, whose own highways comprise only 2 per cent of all highways, control nearly half of the communal roads and draw upon the entire authorized amount of available communal resources for those roads, more than half of which are passably maintained. The communes are thus left with inadequate resources for work on their ordinary vicinal roads (over 50 per cent of the total road systems) and these are in an unsatisfactory state of maintenance, while the remaining "rural" roads have in many cases become practically impassable.

Steps have been taken to effect a reclassification of all highways, taking account of traffic characteristics, which will result in a greater proportion of expense being provided by the National Government. Only 43 per cent of the money derived from motor vehicles in 1928 was devoted to road expenditures, and the report points out that the additional National Government highway expenditures would be approximately met by allocation to this purpose of the remainder of the motor vehicle taxes. It is also pointed out that the total expenditures for highways by all jurisdictions (national, departmental and communal) which amounted to 2,252 million francs is exceeded by the total receipts of 2,300 million francs from all user taxes, including license, fuel, excise, and highway taxes. User taxes are mainly imposed by the National Government but the communes may levy on motor vehicles a tax equal to 17 per cent of the national tax and the departments up to 8½ per cent.

The French report makes but little reference to the use of loans as a means of expediting the provision of needed improved highways.

There is a system of national subsidies to departments and communes. Portions of through highways lying within municipalities are supported by the authorities responsible for such highways.

Germany.—In most of the German States the main highways are maintained by the States themselves; in Prussia by the Provinces. Throughout the country there are also district and town road jurisdictions. Each jurisdiction at present has some heavy, some medium, and some light traffic highways, but a reclassification is in progress to give to the States or Provinces the heavy-traffic highways in order, according to the report, to get them "under the most efficient authorities."

There is little new construction except by cities and towns in new districts and except for a few special roads such as that from Bonn to Cologne and Dusseldorf. Much reconstruction is, however, found necessary to meet increased traffic. It is recognized that the adaptation of the highway system to modern requirements has only begun. Maintenance is a first charge in the budget.

General taxes are the principal source of revenue for highways in Germany. Motor-vehicle taxes will not cover more than the major part of maintenance costs and opposition to increase in the rate of those taxes is reported. They are collected by the National

Government and distributed to the States in fixed proportions and by them to Provinces, districts, and municipalities as decided by the States. They are to be applied solely on roads and bridges. State subsidies from the general treasury are given to Prussian Provinces. Other sources of highway revenues are unemployed relief funds and assessments of benefited property. Tolls are opposed as disturbing traffic.

The report holds that general property and business are undesirably burdened and that it is not possible to make available all the funds required by the construction program. At the present rate 32 years would be required to complete the highway system. Improvement of this condition and supply of the funds needed can only be expected from the negotiation of highway loans which have been tried to a limited extent in most of the German States. In urging such loans the report states that, besides benefiting the motor industry and business in general, a highway system in good condition unquestionably reduces maintenance costs.

Great Britain.—In Great Britain there is no intermediate highway authority between the National Government and the local units. The National Government does not directly engage in highway construction and maintenance, but its Ministry of Transport has important supervisory and advisory powers and its Ministry of Health certain responsibilities as to bond issues of local units and local expenditures on national roads.

Highway revenues come from two main sources—local rates (taxes) levied on the annual value of occupied property, and National Government grants from the road fund. Grants to counties are on a fixed percentage basis of maintenance costs for each class of highway and special grants are given for new construction on a percentage basis. County boroughs (cities) and London authorities receive grants for special construction work but nothing for maintenance. The road fund is derived from general taxes and taxes on the motor vehicle and its use—fuel taxes and registration fees. Motor vehicle revenues are in part used for other than highway purposes but this diversion is criticized. Assessments on benefited property are applied only for street improvements. Tolls are generally being abolished but are being applied in a few major undertakings such as the Mersey Tunnel.

Local construction work or extensive reconstruction and improvement is normally financed by bond issues to be liquidated out of current revenues which likewise take care of all maintenance costs.

The belief is expressed that there should be the widest possible range of contributors to highway revenues, that not less than 25 per cent nor more than 75 per cent of the costs should be met out of general taxation, that the area of charge should be as wide as possible, that the owners of land should make special contributions on the grounds of benefits and that the remainder of the costs should be shared by all users.

Holland.—The report for Holland states that the increase of motor traffic since the World War has demanded a higher standard for most of the roads and has brought out the need for new main roads and for bridges over principal waterways. Superhighways costing 300,000 to 600,000 florins per kilometer and secondary improved roads costing 30,000 to 50,000 florins per kilometer are contemplated.

Highways were reclassified in 1926 into first class (national), second class (provincial), and other (commune and polder). There are grants from the national road fund to the Provinces, whose road schemes are subject to approval by the national authorities. Grants to lesser jurisdictions are for the present given only to promote abolition of toll roads which are regarded as undesirable on account of the cost of toll collection and the diversion of traffic. Fuel and tire taxes are not approved but motor vehicle license taxes provide a large share of the road fund which also receives grants and advances from the general treasury. Motor-vehicle taxes are not to be applied to maintenance of existing roads but they should provide for most of construction and improvement and for any increase in total maintenance costs due to growth of traffic.

Loans are used only for construction and are repayable within 30 to 40 years. Without loans only half the first and second-class systems could be completed in 55 years and it would require 12 years more to finish the work. A calculation has developed that capital invested in provincial roads during the period from 1929 to 1944 inclusive will be entirely redeemed in 1971.

Ireland.—The report from Ireland indicates that its plan of road administration by county and city councils is substantially the same as during the British control. The National Roads Department does not carry on construction or maintenance but distributes grants for the improvement and for a portion of the maintenance of main roads and exercises a degree of supervisory authority. All main roads are reported as in good condition.

Motor-vehicle taxes provide the national road fund, while general taxes are the source of revenue for county roads. Loans have been used to some extent.

Italy.—The Italian report emphasizes the financial difficulties met by Italy and other nations in providing for the enormous growth of highway traffic and describes the complicated relationships, sharing of costs, and distribution of subsidies which have prevailed in Italian highway work. It points out that the interest of the National Government in the main highway system, which was turned over to the Provinces upon the advent of the railways, has revived since the coming of motor transportation. An autonomous national highway administration (A. A. S. S.) was created in 1928 to control the national system of 20,000 kilometers of main roads which carry two-thirds of the total motor traffic. A reclassification of the remaining highways is expected under which the Provinces will provide for secondary and local roads, except in cities and towns, subject to increased supervision and allotment of funds by the A. A. S. S. The report recommends that sections of through roads lying within cities or towns be entirely paid for by the national or provincial government in charge of the particular road, instead of only repaving costs and part of the maintenance costs as at present.

Construction costs are paid out of general revenues and user taxes (license fees) cover only maintenance and improvement. Recently motor-vehicle revenues have been collected solely by the National Government, the Provinces and communes retaining only the right to tax animal-drawn vehicles. The A. A. S. S. has revenues from a variety of sources, including motor-vehicle taxes, fines, surface and subsoil privileges, advertising rights on highways outside cities, levies on industries using the highways, assessments on increment in value of property due to improvements, and annuities (loans).

Assessments have been applied chiefly in cities but provision has been made for their application elsewhere, particularly in reclamation projects (except in southern Italy). Toll roads are generally being eliminated. Loans have been extensively used by Provinces and local jurisdictions.

With regard to the Milan-Italian Lakes autostrade, established as a private toll road with a contingent guaranty of the company's bonds by the Government, the report states that until now it has not been necessary for the Government to pay anything under this guaranty. For other autostrades, including two completed, two under construction and several projected, the Government goes further by making construction loans to the concessionaires. This aid is in addition to outright subsidies given by local administrations. The report finds in these more generous methods no contradiction to the conclusion of the Milan Congress which, the report points out, did not exclude Government subsidies necessary to attract private capital to an enterprise of recognized public utility.

Siam.—The report from Siam shows that highway construction and maintenance are generally financed by the State. Definite provision for maintenance is made in the national budget.

The policy has been to develop feeder roads to the railways and waterways in preference to trunk highways parallel thereto but the ultimate development of a national highway system with local roads under provincial control is envisaged.

Spain.—The specially surfaced roads of Spain were constituted as a national system (Circuito) by royal decree of February 9, 1929, and will be administered by a commission having citizen as well as official representation. The sections of these roads in cities are included in the national system.

Sources of revenue for the Circuito are derived from general taxes, special taxes on vehicles and a share of the license taxes, poll taxes levied against the municipalities, special taxes on motor carriers and revenue from the sale of land along the road. General taxation provides approximately 75 per cent of the highway income and motor vehicle taxes nearly all of the remainder. A very large proportion of improvement and maintenance costs for the next 10 years is to be financed from an advance from the general treasury.

The motor vehicle license taxes are distributed 25 per cent to the National Government, 25 per cent to the Circuito, 15 per cent to the Provinces, and 35 per cent to the municipalities. The need for largely increased funds for highway purposes is recognized in the report which recommends, among other things, fuel, oil, and tire taxes for exclusive application to highway uses. It is regarded as an important principle that the highway users through their organizations should have a voice in the administration of the funds contributed by them.

Sweden.—The report from Sweden recognizes the road problem as one of the most important questions in the country and states that nation-wide improvement of the heavy traffic main roads and the trunk highways of several cities is necessary in order to decrease their annual maintenance costs, increase their traffic capacity and decrease transportation costs. The report develops detailed figures as to the costs of construction and maintenance of different types of highways, showing that for heavy traffic the higher types are more economical and justify extensive capital investments.

United States.—The report from the United States points to the bond plan and the "pay-as-you-go" plan as the two principal methods of financing the cost of highway improvements in the United States, neither method being used in any State to the exclusion of the other. While the bond method involves greater total cost, the income produced by the economic improvement more than offsets the interest cost. The public benefits by reduced motor vehicle operating costs and by quickening of economic and social life. Recent State bond issues are generally retired from motor vehicle taxes, local highway bonds from general property taxes. "Funds raised from bond issues," states the report, "should be expended only for road improvements of a relatively permanent character; the maintenance of the improvement should be assured beforehand; and adequate provision should be made for interest payments on the bonds and for their retirement as early as possible."

The report further declares that bonding should not be resorted to except to the extent that may be necessary if highways can not be built from current funds by the "pay-as-you-go" plan. There is caution against too high type of construction not justified by traffic, especially when bond issues therefor are authorized through popular referendum; also that it is easier for government units to run into debt than to get out of it. Necessary highway improvements, however, under honest and efficient expenditure bring such benefits that either bond or "pay-as-you-go" method, whichever may be necessary for a given improvement, is justified.

The report points out the increasing part of highway income formed by motor-vehicle revenues, being used principally for maintenance and in many cases for interest and amortization of highway bonds. License fees and gasoline taxes are preferred to tolls as a means of collecting contributions from highway users.

The report includes three papers—Financing Highways with Bonds, Highway Finance by the Pay-as-You-Go Plan, and Motor Vehicle Revenues as a Source of Highway Finance—which present much detail.

The first paper shows that highway bonds have been issued by 31 of the 48 States of the United States and by many counties and local units. There are no Federal highway bonds. The bonds are usually of two types, serial or sinking fund, with increased preference for the serial type. Eight outstanding advantages of bond financing are presented, and two inherent objections to the financing of highways solely from current income.

The second paper, while recognizing that some bonding will generally be necessary and advisable, presents the viewpoint of those who advocate following the "pay-as-you-go" plan to the maximum practicable extent and outlines the objections to bond issues.

The third paper presents figures showing the increasing share which motor-vehicle revenues, exclusive of property taxes, contribute toward the total highway expenditures (46 per cent in 1928) and briefly explains the theory of taxing the two groups benefited by highway improvements, i. e., the general public and the highway users.

Attached to the report is an appendix containing a statement of the principles of highway administration and finance adopted by the American Association of State Highway Officials on November 20, 1924. The report states that, while wide variance in highway development in the different States prevents adoption of uniform policies of highway finance, the basic principles contained in the statement, which were developed through experience in the United States, may perhaps be generally applied.

CONCLUSIONS

1. The motor vehicle with increasing use creates demand for large expenditures for reconstruction and improvement of existing highways to new standards, construction of new highways and effective maintenance of all improved highways. It justifies such expenditures through improved economy and efficiency of transportation within areas previously served by highway transport and through extension of economic commercial and social access to new and wider areas.

2. No country has as yet approached completion of its major highway system to the new standards and all countries face heavy demands for greater attention to accommodating their secondary and local roads to the needs of motor traffic. Though differing widely in character and degree in different countries, the problems of highway finance are universal and urgent.

3. To meet the large financial problems involved and to secure the fullest and quickest benefits from the new form of transportation, it is specially important that highways programs covering a period of years be set up well in advance and carefully budgeted. Adjustments to meet changing conditions and improved methods can readily be made as developments may require.

4. To facilitate financing and administration, and as a guide in determining the kind and extent of improvement, all highways should, as far as practicable, be classified according to the characteristics of their prevailing traffic as to origin, destination, and importance. Classifications usually applicable are:

(*a*) General use highways:
  (1) Main or national highways.
  (2) Secondary, departmental, provincial or, in countries of smaller geographical area, county highways.

(*b*) Local highways:
  (1) Local, vicinal, or communal roads.
  (2) Urban streets, including sections of main or secondary highways.

(*c*) Special highways: Military roads, autostrades, etc.

The first group includes the highways of general use, i. e., highways which receive traffic from a number of local roads or from a city or town and carry a considerable proportion of traffic into or through more than one rural jurisdiction. Every public highway should be definitely assigned to the appropriate class, and responsibility therefor should be fixed upon the authorities of the proper political unit.

5. In countries of vast area and sparse population financial limitations and traffic needs may direct primary attention to early development to minimum all-year standards of highways of secondary or local character to give access to rail or water transport lines. As highway development progresses, however, such secondary and local systems tend to integrate and through routes become identified with consequent demand and financial justification for higher type construction. It is important in the interest of ultimate economy that the requirements of the future main highway systems be considered in planning earlier highway development.

6. To promote efficiency of programming and administration highway authorities of higher jurisdictions should have actual supervisory or advisory relationship to the lower jurisdictions. The granting of subsidies or loans by the national to the lower jurisdictions under suitable conditions is an effective means of exercising the desired influence and of assuring financial possibility of carrying out highway programs in charge of lower jurisdictions having aspects of national concern, including special attention to undeveloped sections.

7. Provision for systematic maintenance of all highways after improvement is an essential feature of a sound highway program. If proper types of highways in relation to the character and volume of traffic are provided, maintenance costs should be less than with inadequate highways handling the same traffic. In making provision for maintenace, however, it should be borne in mind that traffic on improved highways tends to increase rapidly and, while the improvement thus benefits the users, it may increase the total of maintenance costs. For this reason maintenance of improved highways of general use, or at least any increase over the former normal maintenance costs, should be regarded as a first charge upon the user revenues.

8. The cost of constructing, improving, and maintaining adequate highway systems should be distributed equitably in relation to the direct and indirect benefits derived, taking into consideration the taxable capacity of those benefited. Wide difference of conditions and institutions in various countries makes it impossible to establish any fixed formula for general application but certain conclusions based on recent trends can be stated:

(*a*) Due to the benefits to society, business, and property in general the application of general tax revenues to highway purposes should continue, the amounts being dependent upon the needs for highways, the funds available and the demands for other purposes in the public budget. General taxes, carrying as they do a direct accountability to public opinion for efficient expenditure, are a particularly appropriate source of revenue for work on local roads, including urban streets.

(*b*) Assessment of abutting property should continue in urban areas but in agricultural areas only in proportion to actual benefit from the degree of improvement useful to such property. Thus where highways have been brought up to the standard needed for agricultural uses, special assessments on such land should where practicable be abandoned. Certain countries have already relieved agricultural lands of all taxation for highway purposes.

(*c*) Up to the limit where they become an undue burden upon users of the highways, user taxes, including license fees and fuel taxes, afford an important and increasing source of highway finance. If such taxes are made unduly high, or if the burdens upon vehicle owners are unnecessarily increased by excessive import duties in predominantly agricultural, nonmanufacturing countries, they tend to bring to bear the law of diminishing returns as well as to deprive the public of the benefits of normal development of motor transportation. For the same reasons user taxes should be applied exclusively for highway purposes. To provide uniformity over reasonable areas they should be imposed only by one of the higher jurisdictions. To maintain the sense of responsibility for collection and expenditure, they should in principle be expended by or under supervision of the collecting jurisdiction and, at least for the present, only on highways of general use (including the sections thereof within municipalities).

9. Because of the prevailing inadequacy of financial resources to permit prompt completion of highway systems corresponding to the economic requirements of motor transportation and because of the investment character of improved highways from which experience shows increasing returns in the form of user taxes may be expected, bond issues for highway construction and improvement are desirable in most countries. Such bond issues should, however, be limited to actual requirements for economically justified construction or improvement projects, under sound administration and with subsequent maintenance provided for out of current revenues. The bonds, if based primarily on user revenues for interest and redemption, which according to experience can be fully covered by such revenues under favorable conditions of highly developed motor transport, should nevertheless be backed by the full credit of the State. Bonds should be serial in form with maturities so arranged that annual requirements of principal and interest will be as nearly uniform as practicable. As more than half the cost of any highway represents grading, drainage, structures, engineering, rights of way, etc., which are of practically indefinite life, and as the other portions have life well beyond the period of amortization thereof under the serial form, the term of bonds for construction of main highways may be 30 years and for improved secondary highways 20 years. When the needs for expedited highway construction and improvement have been met, the use of bond issues should be discontinued and the pay-as-you-go basis followed, especially in cases where capital expenditures recur in each annual budget.

HIGHWAY TRANSPORT: CORRELATION AND COORDINATION WITH OTHER MEANS OF TRANSPORT; ADAPTATION TO COLLECTIVE (ORGANIZATIONS) AND INDIVIDUAL USES

General report submitted by HENRY R. TRUMBOWER, Professor of Economics, University of Wisconsin; Economist, U. S. Bureau of Public Roads

Highway transportation has in the last decade become firmly established in the general scheme of transportation in the important and progressive countries of the world. The people and government agencies of the several countries are beginning to investigate the possibilities of the coordination in the movement of persons and commodities by highway on the one hand and by rail, water, and air on the other. The coordination of rail and highway transportation is the more pressing problem. The development of highway transportation through the use of the motor vehicle has not been of equal intensity in all countries. To the degree that this development has gone on in a country to that extent has the problem of the coordination and correlation of highway and rail transportation facilities become the more pressing in its demand for a solution based upon broad economic and scientific principles so that the public as a whole may enjoy the maximum benefits of all of its transportation agencies.

Transportation by highway and transportation by rail are partly complementary and partly distinct services. Each one must be judged on its own merits. The considerations which govern the one are not the same as the considerations which govern the other. One can not be placed in a subordinate position as compared with the other.

In considering this problem of coordination it must be recognized that common carriers of both passengers and freight constitute but a very small part of the total highway traffic. Private automobiles, in the United States for instance, are handling most of the total travel and have taken from the railroads more than two-thirds of their local passenger business. Where such a condition occurs public authorities should permit the railways to adjust their train schedules so as to reduce passenger train miles as much as possible. Railroads find it advantageous to substitute for unprofitable trains busses owned by them or operated by one of their affiliated companies.

Bus operation, whether carried on by a railway company or by an independent operator, must be conducted as a regulated monopoly. Excessive competition results in less adequate service, with unnecessarily low rates, which aggravate the loss to competing railway lines.

In certain situations it is found that the small amount of traffic that highway common carriers might draw from the railways is largely compensated by the feeder service which they afford to main lines of railroads. This is especially true in mountainous countries where railway construction is extremely expensive. There the automobile, by superseding the old and slow means of transport, has brought about a revolution in traffic and has caused such regions to be better developed industrially and commercially.

In considering the various proposals for a closer coordination between rail and highway carriers, one of the following three plans is usually followed:

1. Voluntary cooperation between railroad companies on the one hand and operators of busses and common-carrier trucks on the other.
2. Inauguration of highway services by the railroad companies or obtaining financial control by the railroads of highway carriers.
3. Quasi-legal coordination with obligation placed on the different transport companies to agree to the creation of a system of cooperative operation and in case agreement is not reached the enforcement of compulsory coordination by governmental authority.

Motor-truck operations present a different problem from that of automobiles and busses because the freight carried by a truck might otherwise be hauled in a train, while passenger cars, including busses, have created an entirely new traffic a part of which railways never could have created or handled. On the other hand, it must be observed, trucks operating over good highways act as agencies for gathering freight which serve to increase the productivity of the farming areas and relieves the railways of short-haul freight upon which little or no profit could be made.

Common-carrier truck operations, as a whole, have not been profitable due to the competition of private and contract trucks. Common carriers of freight on the highways handle such a small part of the total traffic that the field does not appear sufficiently attractive for railways as a general proposition.

The average motor-truck trip is considerably shorter than the average passenger-automobile trip. Few hauls by truck exceed 100 miles and generally three-fourths of all movements are under 30 miles. These results are shown by highway traffic surveys. The situation is quite different with respect to passenger-car operations. Surveys indicate that while nearly or about half of all passenger-car trips are under 30 miles in length, almost if not quite 25 per cent are longer than 100 miles.

Because of the peculiarity of highway traffic being free from specific road charges it is not possible without special effort to measure its volume as is done with railroad and other types of traffic. Estimates based on certain studies and investigations have to suffice.

For the further development of highway transportation that traffic should be financially self-sustaining. Monetary subsidies on the part of the State or of private interests should arise only in the opening up of certain regions or sections of country which are destitute of traffic. Otherwise, the motor vehicle should be in position to bear its own expenses and taxes in so far as they may be considered fair to the motor vehicle. This is true in particular with reference to the contribution for the maintenance of roads which the motor vehicle makes by the payment of motor-vehicle taxes. In this regard, it should be kept in mind that the highways are also at the service of other users and therefore the entire costs can not be imposed on the motor vehicle alone. Taxes should be measured very largely by the use made of highways and should not be such as would arbitrarily prohibit the use of the highways.

Railways and roads are now capable of rendering a service in excess of the traffic requirements of a given area. Cooperation between railroads and automobiles is one of the great requirements of the age. It is, however, much easier to demand a solution of the railroad-automobile problem than to present one.

1. TRAFFIC REGULATION IN LARGE CITIES AND THEIR SUBURBS; TRAFFIC SIGNALS; DESIGN AND LAYOUT OF ROADS AND ADAPTATION TO TRAFFIC REQUIREMENTS IN BUILT-UP AREAS

2. PARKING AND GARAGING OF ROAD VEHICLES

General report submitted by DR. MILLER M'CLINTOCK, Director Albert Russel Erskine Bureau for Street Traffic Research, Harvard University, Cambridge, Mass.

The control of street traffic in urban areas is generally recognized as a problem of great and growing importance, affecting vitally public safety and community economics. Relief is to be found in improved types of regulation and in a better design of physical facilities.

TRAFFIC REGULATION

The physical and financial difficulties involved in street reconstruction make it imperative that chief reliance be placed upon traffic control measures for the improvement of conditions in built-up areas.

Uniformity.—The rapid growth of intercity and international automobile traffic requires that serious attention be given to the question of uniformity in order that confusion may be avoided. Several countries are making definite progress toward national uniformity. A general similarity in the rules of the road would appear to make an international uniform code a practical possibility at the present time.

Basis for special regulations.—The design of traffic regulations to meet special local requirements demands careful investigation. Control measures based on guesswork may have serious economic consequences. Where properly applied, control measures have proved their ability to correct many difficult conditions. The design of regulations should be the responsibility of a competent technical agency of the Government. So far as possible traffic difficulties should be anticipated, studies made and regulations drafted, before conditions become critical.

Types of relief regulation.—For the improvement of conditions in congested districts emphasis is placed upon the use of rules to control pedestrians, to exclude certain types of obstructive traffic, to limit or prohibit parking, to provide better alignment of traffic, and to give the most efficient routing to traffic of all classes.

TRAFFIC SIGNALS

This title has been interpreted as including signs for the purpose of informing, warning and directing traffic, as well as signals used for the purpose of alternating traffic movements at intersections.

Uniformity.—The growing necessity for national and international harmony with respect to the design and use of traffic signs and signals is generally recognized. At present national standards with respect to signs show great variations. There is, however, an underlying harmony in principle. Thus all reporters emphasize the importance of shape, color, and symbols rather than text for the conveying of sign messages.

Practice with respect to traffic control signals shows a definite tendency toward uniformity. While there are several exceptions optical signals using the three colors, red, yellow, and green are generally favored. Flexible progressive timing is noted by several reporters as the most efficient type of coordination for signals located in congested districts.

Application.—The effectiveness of traffic signs and signals depends to a large degree upon the manner in which they are used. Adequate plans can not be designed by inexperienced persons. The function should be exercised by a competent public agency with technical experience.

DESIGN AND LAYOUT OF ROADS

The problems introduced by the automobile both with respect to volume of traffic and characteristics of movement have rendered street systems in built-up areas incapable of performing their services in an effective manner. For a correction of these difficulties emphasis is placed on details of design rather than upon comprehensive replanning.

Cross-section design.—The relation of width to effective traffic capacity is a mooted question. Decisions range from a conclusion that major streets can scarcely be too wide, to a belief that too generous widths have been used in the past. Similarly there is no general consensus of opinion as to proper lane widths.

It is recognized that there is a growing necessity for an accurate distribution of available roadway widths to the various classes of traffic demands and that where possible this division should be accomplished by the character of the design and construction. Traffic streams moving in opposite directions should be separated by a neutral strip. Street car and automobile traffic should be protected from mutual interference by a physical separation of their operating lanes. Where possible, fast and slow traffic should be allocated to different operating areas.

Intersection design.—Several reporters express the belief that typical street plans provide too many intersections along main routes of travel and that future planning must provide less frequent access. All recognize intersection design as a critical problem in traffic difficulties.

Rotary traffic or provision for indirect, roundabout crossing is the relief most frequently proposed. It is generally accepted, however, that at the more heavily traveled intersections an actual separation of roadway grades will be desirable and ultimately necessary. Where a series of such intersections exists in close proximity this proposal leads logically to an entire elimination of grades through an elevation or depression of the main route. In highly congested districts the ultimate development suggested is a separation of rail, automobile, and pedestrian traffic by the use of three distinct levels.

General layout. Planning should be based on a broad view of city and regional requirements substantiated by competent factual studies. In addition to adequate radial streets, the system should be provided with bypass routes around the central congested areas and likewise with highway routes around the entire city so that through traffic may avoid the use of streets required for local traffic.

PARKING AND GARAGING OF ROAD VEHICLES

Parking is generally recognized as a secondary and inferior use of public streets and one which may be restricted or prohibited when it interferes with travel requirements. The elimination of all parking in congested business districts is a definite probability in view of growing traffic demands. The importance of parking from the standpoint of automobile use and business operations makes it imperative that alternative terminal facilities be provided.

Current developments indicate that such facilities will be supplied as demand arises in public or private parking lots or garages.

INTERNATIONAL ROAD CONGRESS ADOPTS GENERAL CONCLUSIONS

THE papers submitted by the general reporters to the Sixth International Road Congress were considered and debated in section meetings on Tuesday, Wednesday, and Thursday of the week of the congress, October 6 to 11. In these meetings the conclusions suggested by the general reporters were discussed, and various revisions and amendments were offered. The conclusions reached their final form in the section meetings of Thursday, and were submitted to the plenary session of the congress held at 2 p. m., Friday, October 10. The conclusions relative to the six questions which constituted the agenda of the congress are here printed in the form in which they were adopted at this plenary session.

QUESTION I-A: RESULTS OBTAINED BY THE USE OF CEMENT

1. Cement is becoming generally used as a paving material and has many inherent advantages. Rapid-hardening cement has special advantages in particular circumstances.

2. Cement has been used successfully in the construction of cement concrete base courses for other surfaces, for cement concrete pavements, and for cement-bound macadam.

3. Cement concrete pavements and also cement concrete base courses protected by appropriate wearing surfaces are suited to heavy traffic.

4. Where a large volume of steel-tired traffic is encountered, if cement concrete is adopted as a pavement material, a 2-course pavement with the upper layer composed of very hard aggregates should be used instead of a single-course pavement.

5. Single-course pavements have successfully carried maximum volumes of traffic and maximum wheel loads when the traffic was largely rubber tired.

6. Cement-bound macadam has been successful on roads carrying light traffic not inimical to the macadam type of construction. This method seems especially advantageous in locations where the condition of drainage or exposure are unfavorable to the use of ordinary water-bound macadam. A protective wearing surface seems equally indispensable on cement-bound and water-bound macadam.

7. In designing cement concrete pavements and cement concrete base courses to be surfaced with other materials the resulting pavements should have equal load-carrying capacity or structural strength, when similar traffic conditions are to be met.

8. Competent engineering supervision of design, construction, and maintenance of cement concrete pavements is necessary to insure good results.

9. It is desirable that subgrades be uniform and stable.

10. Pavement slabs must be designed to carry expected loads. Edge thickening is advantageous as a means of producing an economic and balanced structural design for concrete pavement slabs.

11. Longitudinal and transverse joints are commonly used and must be designed to meet traffic, subgrade, climatic conditions, and shrinkage of concrete, but in view of the fact that a number of concrete roads have been successfully constructed without joints, it is advisable that further research should be made on the whole subject of joints and cracks.

12. Scientific design of concrete mixes and weight proportioning of aggregates represent the most modern practice.

13. Construction operations are performed mostly by machinery, with resulting lower cost and better workmanship. The value of a cement concrete roadway depends to a large extent on the perfect execution of the work and, in particular, on the homogeneity of the concrete.

14. Thorough curing of concrete surfaces is essential.

15. Maintenance of concrete surfaces, properly constructed, is relatively simple and reasonable in cost. In particular, maintenance should comprise immediate filling with suitable material of the expansion joints and of any cracks which may occur.

QUESTION 1-B: RESULTS OBTAINED BY THE USE OF BRICK OR OTHER ARTIFICIAL PAVING

Brick paving.—Subject to suitable foundation, brick on edge will make satisfactory paving for light, medium, or heavy traffic according to the country in which it is used.

Specifications and tests for paving bricks should be prepared and submitted to the next Congress, with a view to standardization.

Rubber paving.—Rubber block paving has had limited application so far. It is, however, a silent paving and suitable for certain positions in large towns.

Research should therefore be continued in the following directions:

(*a*) Most suitable quality of rubber for street paving.

(*b*) Most suitable type of block and method of laying.

(*c*) Production and method of application of a suitable joining material.

(*d*) Reduction of costs.

QUESTION 2: THE MOST RECENT METHODS ADOPTED FOR THE USE OF TAR, BITUMEN, AND ASPHALT IN ROAD CONSTRUCTION

1. Tar, bitumen (asphaltic bitumen), and asphalt are suitable materials for use in the improvement of all classes of roads, subject to the limitations imposed by the characteristics of the material, the intensity of traffic, and local physical conditions. It is especially desirable to note the extended use in recent years of superficial coatings, in particular those effected with emulsions. Certain governing conditions generally recognized are:

(*a*) The use of the correct grade and amount of bituminous binder for the particular type of work and aggregates to be used.

(*b*) Suitable aggregates, correctly graded.

(*c*) Provision for prompt maintenance. When the foundation is adequate and when the surface can be maintained at reasonable cost, an advantage of the types of roads built with these materials is that they can be kept in good condition by maintenance of the surface without the need of extensive repairs to the foundation.

(*d*) Proper porportions of the constituent materials, thorough mixing and thorough compacting.

(*e*) Competent technical supervision of design, construction, and maintenance.

(*f*) Slipperiness. Attention should be given to the reduction of slipperiness. In some cases valuable results have been attained by the following methods:

(1) Adoption of suitable compositions containing the maximum permissible proportions of large aggregate.

(2) The rolling of plain or precoated chippings into newly laid surfaces.

(3) The use of minimum practicable camber of surface and proper degree of superelevation on curves.

(4) Treatment of existing surface with a suitable surface dressing compound, covering with coarse hard chippings, and rolling.

2. The need for research into the fundamental factors involved in the use of tar, bitumen, and asphalt in road improvement and maintenance is recognized. The attention of the Members of Congress is especially directed to needs as follows:

(*a*) Research into the constitutions and characteristics of bituminous materials and their combinations with other materials, with proper study of their application in the treatment of earth roads.

(*b*) Improvement of mechanical equipment for use with these materials and their combinations with mineral aggregates.

(*c*) Research into factors affecting the durability and serviceability of roads, such as climate, subsoil, traffic density, and road design.

(*d*) Data on economic factors, such as—

(1) Cost of transportation over various road types, including vehicle operation costs and roadway costs, with proper study of their application in the treatment of earth roads.

(2) Relation between maintenance costs and intensity of traffic for various road types.

3. In order that mutual understanding and helpfulness may be promoted, there is need for an international classification of materials, combinations of materials, construction methods, and road types which will correlate the terms in use throughout the world, both commercial and scientific.

QUESTION 3: THE CONSTRUCTION OF ROADS IN NEW COUNTRIES, SUCH AS COLONIES AND UNDEVELOPED REGIONS

1. It is desirable, in the first place, to form a central body with legal authority to coordinate and assist the general planning of the road communications of a country and the acquisition or reservation of necessary land.

2. The perfection which has been attained in the building of automotive vehicles often permits highways to render service in the exploration and development of new territories which formerly could be rendered only by railways.

3. The highway has over the railway the advantage of allowing the expense of construction and maintenance to be made proportionate to the importance of the traffic to be served. The present-day automobile being able to run on very difficult roads, at the beginning we can establish simple trails by nothing more than a primitive working over of the natural soil, the structures which are necessary being only those over permanent watercourses which can not be forded. The road will ultimately be improved by the construction of a roadway provided with an appropriate surfacing, and the construction of structures for crossing streams and valleys in proportion as the development of traffic makes available new resources for the execution of the work.

4. Adequate land should be acquired to provide for the eventual and ultimate expansion of the traffic as far as can be foreseen.

5. Before any actual construction is begun it is essential that the general highway layout should be prepared with due regard to the ultimate requirements of the permanent road system.

6. In countries of low density of population and where the construction of a large mileage of roads adequate even for light traffic is limited by the scarcity of available funds it is deemed advisable to adopt the progressive system of construction by stages. Every endeavor should be made to insure that the initial alignment, grading, and subsequent surfacing should

be such that all work executed should be capable of being utilized in the ultimate development of the road structure.

7. In the first instance the actual traveled way should be so constructed as to be passable for motor traffic, but as economically as possible.

8. Locations in cut should be avoided when they have disadvantages from the point of view of drainage, and light fills are to be preferred.

9. Where traffic is light in numbers and unit weight earth roads have proved economical and satisfactory, but to conserve the surface of such roads until they can be improved, it is essential to restrict the weight per unit width of tire and speeds of individual vehicles to such an extent as may be necessary to prevent undue erosion or disintegration of the road surface.

10. For the establishment of roadways a width of 3 meters per traffic lane should be required. On structures a width of 3 meters, or even multiples of 3 meters, should be reserved for traffic.

It is recommended that from the beginning two-lane roadways be provided on important structures, at least in so far as the foundations are concerned.

11. Uniformly of design throughout the length of a location should be attained by avoiding heavy grades and sharp curves in stretches of road which in general do not contain them.

12. Some tests of mechanical grading of the roads have been made in desert regions; it is desirable that these tests be continued.

13. It is desirable also that systematic research be undertaken to determine the physical properties of mixtures of clays and sands which constitute the natural soils including soils containing hygroscopic salts, with a view to collecting useful data for the construction of earth wearing surfaces, appropriate for economic service in little developed regions having available few resources.

QUESTION 1: WAYS AND MEANS OF FINANCING HIGHWAYS: (A) ROAD CONSTRUCTION; (B) MAINTENANCE

1. The motor vehicle with increasing use creates demand for large expenditures for reconstruction and improvement of existing highways to new standards, construction of new highways and effective maintenance of all improved highways. It justifies such expenditures through improved economy and efficiency of transporation especially within areas previously served by highway transport and through extension of economic, commercial, and social access to new and wider areas.

2. No country has as yet approached completion of its major highway system to the new standards and all countries face heavy demands for greater attention to accommodating their secondary and local roads to the needs of motor traffic. Though differing widely in character and degree in different countries, the problems of highway finance are universal and urgent.

3. To meet the large financial problems involved and to secure the fullest and quickest benefits from the new form of transportation, it is specially important that highway programs covering a period of years be set up well in advance and carefully budgeted. Adjustments to meet changing conditions and improved methods can readily be made as developments may require.

4. To facilitate financing and administration, and as a guide in determining the kind and extent of improvement, all highways should, as far as practicable, be classified according to the characteristics of their prevailing traffic as to origin, destination, and importance. Classifications usually applicable are:

- (*a*) General-use highways (including urban streets which form part of such highways):
  - (1) Primary or national highways.
  - (2) Secondary, departmental, provincial or, in countries of smaller geographical area, county highways.
- (*b*) Highways of local interest:
  - (1) Local roads.
  - (2) Urban streets (except as indicated in subparagraph (*a*) above).
- (*c*) Special highways: Military roads, autostrades, etc.

The first group includes the highways of general use, i. e., highways which receive traffic from a number of local roads or from a city or town and carry a considerable proportion of traffic into or through more than one rural jurisdiction. Every public highway should be definitely assigned to the appropriate class and responsibility therefor be fixed upon the authorities of the proper political unit.

5. In countries of vast area and sparse population financial limitations and traffic needs may direct primary attention to early development to minimum all-year standards of highways of secondary or local character, to give access to rail or water transport lines. As highway development progresses, however, such secondary and local systems tend to integrate and through routes become identified, with consequent demand and financial justification for higher type construction. It is important in the interest of ultimate economy that the requirements of the future main highway systems be considered in planning earlier highway development.

6. To promote efficiency of programming and administration highway authorities of higher jurisdictions should have supervisory or advisory relationship to the lower jurisdictions. The granting of subsidies or loans by the national to the lower jurisdictions under suitable conditions is an effective means of exercising the desired influence and of assuring financial possibility of carrying out highway programs in charge of lower jurisdictions having aspects of national concern, including special attention to undeveloped sections.

7. Provision for systematic maintenance of all highways after improvement is an essential feature of a sound highway program. If proper types of highways in relation to the character and volume of traffic are provided, maintenance costs should be less than with inadequate highways handling the same traffic. In making provision for maintenance, however, it should be borne in mind that traffic on improved highways tends to increase rapidly and, while the improvement thus benefits the users, it may increase the total of maintenance costs. For this reason maintenance of improved highways of general use, or at least any increase over the former normal maintenance costs, should be regarded as a first charge upon the user revenues.

8. The cost of constructing, improving, and maintaining adequate highway systems should be distributed equitably in relation to the direct and indirect benefits derived, taking into consideration the taxable capacity of those benefited. Wide difference of conditions and institutions in various countries makes it impossible to establish any fixed formula for general application, but certain conclusions based on recent trends can be stated:

(*a*) Due to the benefits to society, business, and property in general the application of general tax revenues to highway purposes is desirable and should continue, the amounts being dependent upon the needs for highways, the funds available, and the demands for other purposes in the public budget. General taxes, carrying as they do a direct accountability to public opinion for efficient expenditure, are a particularly appropriate source of revenue for work on local roads, including urban streets.

(*b*) Any assessment of abutting or other benefited property, chiefly in urban districts and their environs, should be proportional to the actual benefit to such property.

(*c*) Up to the limit where they become an undue burden upon users of the highways, user taxes, including license fees and fuel taxes, afford an important and increasing source of highway finance. If such taxes are made unduly high, or if the burdens upon vehicle owners are unnecessarily increased by excessive import duties in predominantly agricultural, nonmanufacturing countries, they tend to bring to bear the law of diminishing returns as well as deprive the public of the benefits of normal development of motor transportation. For the same reasons user taxes should be applied exclusively for highway purposes. To provide uniformity over reasonable areas they should be imposed only by rules fixed by one of the higher jurisdictions. To maintain the sense of responsibility for collection and expenditure, they should in principle be expended under supervision of the collecting unit of government, and, at least for the present, only on highways of general use (including the sections thereof within municipalities).

9. Because of the prevailing inadequacy of financial resources to permit prompt completion of highway systems corresponding to the economic requirements of motor transportation, and because of the investment character of improved highways from which experience shows increasing returns in the form of user taxes may be expected, bond issues or other methods of borrowing for highway construction and improvement are desirable in most countries. Such bond issues should, however, be limited to actual requirements for economically justified construction or improvement projects, under sound administration and with subsequent maintenance provided for out of current revenues. The bonds, if based primarily on user revenues for interest and redemption, which according to experience can be fully covered by such revenues under favorable conditions of highly developed motor transport, should nevertheless be backed by the full credit of the unit of government concerned. The period of amortization of loans for highway construction should not exceed the life of the improvement. When the needs for expedited highway construction and improvement have been met, the use of bond issues should be discontinued and the pay-as-you-go basis followed, especially when capital expenditures recur in each annual budget.

QUESTION 5: HIGHWAY TRANSPORT; CORRELATION AND COORDINATION WITH OTHER METHODS OF TRANSPORT; ADAPTATION TO COLLECTIVE (ORGANIZATIONS) AND INDIVIDUAL USES

1. Highway transportation has in the last decade become firmly established in the general scheme of transportation in the important and progressive countries of the world. The people and government agencies of the several countries are beginning to investigate the possibilities of the coordination in the movement of persons and commodities by highway on the one hand and by rail, water, and air on the other. The coordination between different systems of transportation by land, by water, and by air should be so arranged that every transport should be done, as far as possible, through the most economical way and that most fitted to the particular needs. In this matter, the public authorities should adopt such legal and fiscal regulations as not to disturb the natural economic conditions of each transportation system.

2. The coordination of rail and highway transportation is the more pressing problem.

3. The development of highway transportation through the use of the motor vehicle has not been of equal intensity in all countries. To the degree that this development has gone on in a country, to that extent has the problem of the coordination and correlation of highway and rail transportation facilities become the more pressing in its demand for a solution based upon broad economic and scientific principles so that the public as a whole may enjoy the maximum benefits of all its transportation agencies.

4. Transportation by highway and transportation by rail are partly complementary and partly distinct services. Each one must be judged on its own merits. The considerations which govern the one are not the same as the considerations which govern the other. One can not be placed in a subordinate position as compared with the other.

5. In considering this problem of coordination it must be recognized that common carriers of both passengers and freight constitute but a very small part of the total highway traffic. In general, private automobiles form the most important part of highway traffic and it is they which compete most seriously with the railways in passenger traffic. Where such a condition occurs public authorities should permit the railways to adjust their train schedules so as to reduce passenger train-miles as much as possible. Railroads find it advantageous to substitute for unprofitable trains busses operated by them or others.

6. The operation of all Public motor omnibus services, irrespective of ownership, must be subject to adequate control by a responsible authority embracing a wide area so as to insure regularity, efficiency, and adequacy of service, safety of the public at large, and avoidance of excessive competition and uneconomic fares.

7. In certain situations it is found that the small amount of traffic that highway common carriers might draw from the railways is largely compensated by the feeder service which they afford to main lines of railroads. This is especially true in mountainous countries where railway construction is extremely expensive. There the automobile, by superseding the old and slow means of transport, has brought about a revolution in traffic and has caused such regions to be better developed industrially and commercially.

8. In considering the various proposals for a closer coordination between rail and highway carriers, one or more of the following three plans is usually followed:

(*a*) Voluntary cooperation between railroad companies on the one hand and operators of busses and common carrier trucks on the other.

(*b*) Inauguration of highway transport services by the railroad companies or financial and administrative control or participation exercised by these companies in the conduct of highway transport undertakings.

(*c*) Quasi legal coordination with obligation placed on the different transport companies to agree to the creation of a system of cooperative operation and in case agreement is not reached the enforcement of compulsory coordination by governmental authority.

9. Automobile and bus operation, as well as motor-truck operation, produces new traffic, part of which the railroads could not handle, and which is enormously helpful in moving shipments of less than carload, and by introducing the use of containers, and thus helping to solve transportation problems between terminals in large cities. The passenger traffic created is both short and long haul, but the motor-truck traffic is in general short haul. It must be observed, trucks operating over good highways act as agencies for gathering freight which serve to increase the productivity of the farming areas and relieve the railways of short-haul freight upon which little or no profit could be made.

10. Common-carrier truck operations, as a whole, have not been profitable due to the competition of private and contract trucks. Common carriers of freight on the highways handle such a small part of the total traffic that the field does not appear sufficiently attractive for railways as a general proposition.

11. Traffic surveys, including studies of origin and destination of traffic, are of special value in revealing the true characteristics of various kinds of motor traffic and their relationship to other forms of transportation, whether as feeders thereto or supplementary thereof.

12. Highway transportation enterprises should be financially self-sustaining. Monetary subsidies on the part of the State or of private interests should arise only in the opening up of certain regions or sections of country which are destitute of traffic. Otherwise, the motor vehicle should be in position to bear its own expenses and taxes in so far as they may be considered fair to the motor vehicle. This is true in particular with reference to the contribution for the maintenance of roads which the motor vehicle makes by the payment of motor vehicle taxes, including gasoline taxes or duties.

13. Taxes for highway purposes should be borne not only by motor vehicles but by all interests which benefit from the highway system, and should not be such as would arbitrarily prohibit the use of highways.

14. Cooperation between railroads and automobiles, which has already been effected to some extent, is one of the great requirements of the age. In seeking such solutions the need of aviation by the provision of airdromes and roads leading to them must not be overlooked.

15. It is desirable for the convenience of the traveling public that there should be standard dates (with the minimum of exceptions) for new time schedules in public-road services, and that there should be universal as well as regional roadway time-tables.

16. The congress, considering that the question relative to the coordination of the various methods of transportation has received formal consideration at the International Congress of Railroads held in Madrid, May 5–15, 1930, under the title "Competition Between Automobile Transportation Systems and Railroads."

Not being able to proceed to the thorough study required by an examination of the conclusions of the Madrid Congress;

Resolves that hereafter the question of establishing coordination and harmony between the various systems of transportation by road, by rail, by waterways, and in the air be considered by the various international congresses that may be called upon to deal with it, and that reports be prepared by joint commissions of the accredited representatives of these various systems of transportation.

QUESTION 6: 1. TRAFFIC REGULATION IN LARGE CITIES AND THEIR SUBURBS; TRAFFIC SIGNALS; DESIGN AND LAYOUT OF ROADS AND ADAPTATION TO TRAFFIC IN BUILT-UP AREAS. 2. PARKING AND GARAGING OF VEHICLES

1. The congress confirms in general the conclusions of the congress of Milan on the fifth question relating to the layout of cities with regard to convenience and safety of traffic.

2. With respect to traffic signs and signals the congress urges the necessity for uniformity and adherence to the principle that shape and color shall be utilized to give indications.

(*a*) The congress recognizes the recommendations of the diplomatic conference held in Paris in 1926 and set out in Bulletin No. 57, May–June, 1928, as an important step toward this end, and it proposes that countries that have not accepted these recommendations should in designing systems of signs give due consideration to the principles contained in that bulletin. And the congress further proposes that an International committee be named by the permanent commission and the Executive Bureau of the Association of International Road Congresses to consider methods for the universal application of the principles contained therein.

(*b*) It is further recommended that the same international committee undertake to propose uniform standards for traffic control signals and other control devices. Pending the establishment of such standards, it is recommended that the color red in traffic control signals be used only for the purpose of stopping traffic; for other traffic indications, such as those marking obstructions in the roadway, the color red may still be used to indicate caution.

3. The Congress recognizes that the design of rules and regulations for the facilitation of traffic in congested districts is a problem of growing complexity and that specific control measures should be applied only after a competent study of local conditions by qualified officials and the cooperation of interests affected. Under suitable conditions the following types of regulations have been found useful:

(*a*) Parking restrictions through the application of space and time limits or prohibitions.

(*b*) Segregation of types of traffic through the exclusion of certain classes of vehicles.

(*c*) The regular alignment of vehicles en route through the use of traffic lane markings.

(*d*) One-way movement.

(*e*) Rotary movement at intersections where center islands of sufficient size and adequate visibility to permit easy rotation can be reserved.

(*f*) Control of turning movements at intersections and of U-turns between intersections.

(*g*) Pedestrian regulation.

4. The Congress recognizes the physical and financial difficulties involved in replanning congested and built-up districts in large cities. It believes, however, that substantial relief can be obtained through changes looking toward the adaptation of streets in such districts to the requirements of modern traffic. Among such adjustments are the following:

(*a*) Where economically possible public rail carriers should be removed from the street surface in such districts and placed in subways, or rapid transit, or forms of transportation offering a minimum obstruction to traffic should be substituted, thus providing improved transportation and an increase in general traffic capacity.

(*b*) The passage of pedestrians across heavily traveled streets can be facilitated and protected through the construction of subways or bridges at street intersections or other natural places of crossing. In certain districts it may be desirable that such subways or bridges be sufficiently close together so that any crossing of the street surface by pedestrians will be rendered unnecessary. Where traffic is not sufficiently heavy to warrant such structures pedestrian traffic can be facilitated and protected by the use of definitely marked lanes at street intersections and other natural places of crossing.

(*c*) In order that prohibition or progressive restriction of parking may be applied without undue public inconvenience or economic hardship encouragement should be given to the provision of offstreet storage space economically and conveniently available. The congress holds that in certain cases it may be proper to require in the construction or remodeling of buildings the incorporation of suitable space for the offstreet loading or unloading and garagings of vehicles.

(*d*) The congress holds that traffic congestion and the resultant risk of accidents as well as economic losses are sufficiently great in certain instances to warrant consideration of expenditures for the construction of grade separation at intersections and indeed for the construction of elevated or underground streets.

5. As regards sections of the city in process of development, and suburban zones destined for future development, the congress urges the application of broadly conceived plans for their layout in order that the future may not result in a repetition of the difficulties now experienced in congested districts.

6. The congress holds that highway officials should give due regard to the amenities of the roadside and should be given such powers as may be necessary to give reasonable protection in connection therewith to highway safety and the recreational value of the road.

BUREAU OF PUBLIC ROADS ENTERS UPON NEW RESEARCH PROGRAMS

ORIGIN AND DESTINATION SURVEY OF MICHIGAN STATE HIGHWAYS BEGUN

In an effort to determine how much of the cost of local roads in Michigan should be paid by local taxpayers and how much should be spread over the State, the Bureau of Public Roads, in cooperation with the Michigan State Highway Department, is conducting an origin and destination traffic survey on all roads, local as well as main State highways, in sample townships in the 83 counties in the State.

The facts obtained by the survey will show to what extent the local roads of each local taxing jurisdiction are used by traffic originating within and without these jurisdictions; and will serve as a basis for the distribution of public funds now being made available for highway improvement. The survey will also determine the amount of tourist traffic in the State.

This investigation was begun on July 1 and will continue for one year, after which time the same agencies will make special studies for one month, in seven cities, to determine the relative use of city streets by city vehicles and by vehicles owned outside the cities. These studies will also be used as a basis for taxation. The cities selected are Ann Arbor, Detroit, Flint, Grand Rapids, Jackson, Lansing, and Niles.

DESIGN OF CONCRETE SLABS BEING STUDIED

To determine the relative efficiency of concrete pavements of several designs and to develop a more exact knowledge of the amount and distribution of stress in pavement slabs resulting from loads applied to them at different points, the bureau has constructed a number of full-size concrete pavement slabs at the Experiment Farm of the Department of Agriculture at Arlington, Va. The slabs will later be subjected to an elaborate series of tests.

A number of the slabs are of uniform thickness throughout. These are expected to give information regarding the relation between loads applied at various points and the stress and strain of the concrete at all points in the loaded cross section, and the relation between load resistance and slab thickness.

Other slabs are thickened at the edges and for a certain distance from the edges in accordance with the different designs used at present in several States. In some cases the thickening is provided for by excavating the subgrade under the edges of the slab. In others the surface of the concrete slab will be raised at the edges so as to form a low, rounded lip curb. In still others the lip curb is combined with a thickening of the edge of the slab at the bottom. Observations of these sections will give information regarding the relative load-resisting properties of designs now in use in various States.

The test slabs, which are 20 feet wide by 40 feet long, have central longitudinal and transverse joints, and one of the objects of the tests is to determine the efficiency of various methods in use for transferring load across these joints.

Other tests to be made include bond tests of dowel bars to determine the length of embedment necessary, the measurement of subgrade friction, with particular attention to the effect of edge thickening of transverse joints on resistance to the sliding of the slabs on the supporting surface, and the measurement of the movement of the slab in the subgrade as a result of temperature and moisture changes.

The work of constructing the slabs was begun early in July and is now completed. Preliminary strength tests will be conducted at the end of the 28-day curing period. Measurements of stresses caused by shrinkage were begun during the latter part of September.

ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publiactions in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Deaprtment's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.
220D. Road Models.
257. Progress Report of Experiments in Dust Prevention and Road Preservation, 1914.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
386D. Public Road Mileage and Revenues in the Middle Atlantic States, 1914.
387D. Public Road Mileage and Revenues in the Southern States, 1914.
388D. Public Road Mileage and Revenues in the New England States, 1914.
390D. Public Road Mileage and Revenues in the United States, 1914. A Summary.
*407D. Progress Reports of Experiments in Dust Prevention and Road Preservation, 1915. 10c.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
1216D. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 94C. T. N. T. as a Blasting Explosive.
331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
*93M. Direct Production Costs of Broken Stone. 25c.
109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National Forest Roads and Trails.

MISCELLANEOUS PUBLICATIONS

No. 76. The Results of Physical Tests of Road-building Rock.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *914Y. Highways and Highway Transportation. 25c.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Connecticut.

Report of a Survey of Transportation on the State Highway System of Ohio.

Report of a Survey of Transportation on the State Highways of Vermont.

Report of a Survey of Transportation on the State Highways of New Hampshire.

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio.

Report of a Survey of Transportation on the State Highways of Pennsylvania.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

SEPTEMBER 30, 1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage	Total			Initial	Stage[1]	Total		
Alabama	2,185.0	$ 1,423,981.94	$ 698,828.28	66.0	8.2	74.2	$ 233,568.87	$ 74,518.46	9.3	3.4	12.7	$ 5,735,788.14	Alabama
Arizona	841.1	3,801,843.02	2,877,895.34	115.3	156.1	271.4	126,418.69	85,180.62	7.4		7.4	3,357,931.31	Arizona
Arkansas	1,758.3	6,292,192.35	2,941,584.25	201.7	46.9	248.6	1,540,747.32	754,871.24	41.9	14.0	55.9	3,048,828.19	Arkansas
California	1,893.8	8,921,624.43	3,517,771.56	164.8	28.0	182.8	1,731,506.38	992,791.87	80.2		80.2	4,565,455.56	California
Colorado	1,189.0	5,189,778.72	2,754,447.59	190.8	81.3	272.1	470,188.40	242,648.29	8.7	18.5	27.2	4,381,212.14	Colorado
Connecticut	243.3	2,595,633.60	1,099,282.57	14.0		14.0	1,102,981.71	299,475.70	14.1		14.1	1,275,603.64	Connecticut
Delaware	265.1	999,111.75	484,599.37	48.7		48.7	236,837.50	115,300.75	13.1		13.1	513,398.34	Delaware
Florida	533.8	4,357,595.12	2,067,901.37	68.7	5.5	74.2	204,596.52	102,348.26	2.8		2.8	2,952,084.34	Florida
Georgia	2,535.2	5,513,030.85	3,083,753.12	166.1	121.7	287.8	751,502.62	375,751.30	27.9	13.0	40.8	5,443,898.25	Georgia
Idaho	1,195.8	2,392,533.19	1,415,770.75	135.6	65.5	202.1	579,023.40	404,000.00	41.9	9.0	50.8	1,886,881.68	Idaho
Illinois	2,029.5	21,878,528.25	9,751,867.59	600.2	59.8	550.0	5,457,495.96	2,540,375.38	173.2	12.5	185.7	8,918,559.70	Illinois
Indiana	1,501.1	4,560,726.70	2,213,573.73	143.5		143.5	903,198.23	375,647.33	26.2		26.2	5,300,374.24	Indiana
Iowa	3,020.3	5,293,199.10	2,874,599.94	53.8	160.5	214.1	105,290.11	1,892.77		4.1	4.1	3,342,943.56	Iowa
Kansas	2,868.8	5,865,333.73	2,731,414.41	269.7	42.9	312.5	581,641.94	289,831.92	25.7	64.4	90.1	4,725,772.35	Kansas
Kentucky	1,351.3	8,020,105.08	3,477,907.07	165.2	182.4	347.6	1,647,448.83	721,196.77	35.8	56.3	102.1	2,158,515.56	Kentucky
Louisiana	1,389.1	4,227,372.00	2,044,306.59	158.0	20.8	179.9	338,235.46	157,517.68	10.6		10.5	2,821,022.08	Louisiana
Maine	537.2	3,613,823.09	1,345,387.65	88.1	1.5	89.7	578,520.39	285,808.09	.8		.8	1,995,257.84	Maine
Maryland	537.4	2,186,715.86	1,079,444.71	47.5	15.4	52.9	207,155.30	51,951.50	1.7	1.5	3.2	1,052,567.00	Maryland
Massachusetts	675.5	5,763,730.11	1,719,020.60	73.0	2.5	75.5	129,562.90	64,831.45	.3		.3	3,462,764.35	Massachusetts
Michigan	1,675.7	8,384,077.03	3,643,097.12	195.9	23.9	218.8	3,564,138.17	1,359,938.00	92.5	5.0	97.5	5,062,062.85	Michigan
Minnesota	3,938.3	11,265,966.46	4,122,953.71	223.7	263.1	486.8						3,512,038.03	Minnesota
Mississippi	1,839.7	539,084.13	319,245.57	36.1	7.7	43.8	319,042.64	159,521.27		13.5	13.5	5,592,293.28	Mississippi
Missouri	2,485.7	9,390,230.00	3,197,589.58	147.4	70.4	217.8	2,805,400.38	1,047,806.86	48.8	22.4	71.0	3,960,764.41	Missouri
Montana	1,791.5	7,529,443.42	4,247,199.90	540.1	58.1	598.2	1,622,711.98	892,903.64	151.2	26.8	178.0	5,118,410.38	Montana
Nebraska	3,682.8	5,953,887.70	3,199,958.77	288.5	156.0	464.5	760,799.51	294,514.62	17.4	39.8	57.0	4,592,523.07	Nebraska
Nevada	1,148.3	1,307,158.79	1,139,533.99	23.2	182.0	215.2	138,434.18	121,105.48	12.8	1.3	13.8	2,057,587.07	Nevada
New Hampshire	353.7	2,044,130.73	711,604.89	44.8	1.5	48.4	81,583.72	23,581.59	.1		.1	609,375.00	New Hampshire
New Jersey	525.7	4,909,506.35	1,209,046.32	48.8		48.8						2,547,196.76	New Jersey
New Mexico	1,830.0	3,814,042.23	2,569,046.44	203.3	56.3	259.5	712,070.87	501,076.98	14.7	1.7	16.4	2,432,574.88	New Mexico
New York	2,514.8	25,632,497.76	5,112,265.00	341.2		341.2	5,414,820.00	1,190,122.50	82.3		82.3	12,520,578.54	New York
North Carolina	1,841.9	4,422,488.49	2,153,801.06	161.8	28.8	190.7	215,495.42	84,433.78	23.5	4.8	28.4	4,784,268.94	North Carolina
North Dakota	4,357.0	2,244,309.44	1,155,395.71	337.2	168.8	505.8	1,068,873.70	534,192.97	122.5	157.1	279.8	2,872,713.86	North Dakota
Ohio	2,271.1	21,661,326.36	7,317,735.45	395.5	35.1	430.8	2,099,233.50	703,234.17	20.8	.8	21.7	4,593,489.21	Ohio
Oklahoma	1,875.3	5,887,078.98	2,598,204.75	195.9	85.3	281.2	555,163.93	265,375.89	10.5	12.0	22.5	3,017,320.73	Oklahoma
Oregon	1,151.2	8,234,903.47	3,573,751.75	256.3	85.2	341.5	143,535.04	89,321.02	14.8		14.8	2,068,408.08	Oregon
Pennsylvania	2,449.1	18,524,518.15	5,662,438.04	232.5		232.5	1,067,334.04	405,564.06	12.5		12,5	5,640,675.45	Pennsylvania
Rhode Island	200.2	2,051,112.63	815,911.64	32.1		32.1	200,206.21	100,103.10	3.7		3.7	519,426.42	Rhode Island
South Carolina	1,861.7	5,300,990.82	2,071,954.52	110.0	97.1	207.1	895,711.34	352,064.20	5.8	18.5	25.1	1,774,814.24	South Carolina
South Dakota	3,810.0	3,738,529.35	1,969,648.14	327.1	109.7	436.8	1,240,575.96	792,318.89	135.3	87.1	202.4	2,474,878.73	South Dakota
Tennessee	1,262.8	5,060,630.69	2,281,631.75	204.5	48.5	253.0	547,829.01	271,884.22	17.7		17.7	3,577,439.88	Tennessee
Texas	5,877.1	13,224,668.90	5,433,537.44	402.0	174.8	575.8	3,464,353.45	1,409,642.26	160.1	61.8	222.0	10,878,570.81	Texas
Utah	990.8	1,323,215.90	954,148.84	86.2	10.5	95.8	538,156.73	397,048.50	44.8	47.8	92.7	1,841,003.60	Utah
Vermont	264.7	2,018,787.08	651,008.71	39.1	2.8	41.7						624,276.71	Vermont
Virginia	1,506.7	5,042,851.57	2,404,014.94	211.8	9.7	221.3	707,735.74	295,194.20	28.4	3.0	29.4	2,581,842.72	Virginia
Washington	947.8	3,646,888.06	1,607,800.00	80.7	36.3	117.0	403,145.13	137,900.00	5.4		5.4	3,008,091.86	Washington
West Virginia	717.1	4,077,873.39	1,464,206.58	81.0	33.5	124.5	1,236,248.30	490,887.70	36.0	7.7	43.7	1,358,531.59	West Virginia
Wisconsin	2,255.2	8,030,206.39	3,719,751.29	212.7	65.1	277.8	1,179,243.18	499,200.00	31.0	3.8	34.9	2,977,034.41	Wisconsin
Wyoming	1,665.1	2,815,729.45	1,869,521.35	206.5	131.2	337.7	251,811.39	164,482.48	18.0	23.5	42.5	1,761,439.50	Wyoming
Hawaii	41.2	863,565.90	359,458.43	21.5		21.5	226,820.16	113,414.57	8.2		8.2	1,852,769.59	Hawaii
TOTALS	84,815.7	306,026,568.29	125,737,130.39	8,458.8	2,961.4	11,420.2	48,596,898.23	20,673,060.29	1,639.9	726.4	2,366.3	172,500,557.85	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

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UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 9 ▽ NOVEMBER, 1930

ATTRACTIVE SCENE ON NEW YORK STATE HIGHWAY SYSTEM

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

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VOL. 11, NO. 9 NOVEMBER, 1930 G. P. St. CLAIR, Edito

TABLE OF CONTENTS

Page
Cooperative Survey of Corrugated Metal Culverts on the Austin–San Antonio Post Road . 173
Methods for the Measurement of Water for Cement Briquet Tests 182
Results Obtained by the Use of Cement. Report of American Engineers to the Sixth International Road Congress 184

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COOPERATIVE SURVEY OF CORRUGATED METAL CULVERTS ON THE AUSTIN-SAN ANTONIO POST ROAD

Reported by **E. F. KELLEY**, Chief, Division of Tests, U. S. Bureau of Public Roads

THE corrugated metal pipe culverts included in this inspection were installed in the spring and summer of 1915 on the Austin-San Antonio Post Road (now U. S. Route 81) in Comal and Hays Counties, Tex. The installations were made as an experiment for the purpose of comparing, under the conditions prevailing in this locality, the behavior of different base metals used in the construction of corrugated sheet metal culverts.

From time to time during the ensuing years inspections have been made by representatives of the Bureau of Public Roads, the last one prior to this survey having been made in 1926. A number of inspections have also been made by representatives of other interested organizations.

From the beginning a very considerable interest has been manifested in this experimental culvert installation. This has been due to the desire for information regarding the relative merits of the base metals used in corrugated culverts, coupled with the fact that this project is unique in having a fairly complete history, and includes a number of base metals of known composition and age.

The only previous published report on this project was issued in 1918.[1] This report gives a general description of the installation, analyses of the base metals, and the results of a condition survey made in March, 1917. Since this survey was made when the culverts had been in place less than two years, deterioration had not progressed sufficiently to be significant. Although no other official report has ever been released, conflicting reports regarding the relative merits of the different base metals, as evidenced by these culverts, have been current.

NEED FOR CAREFUL SURVEY REALIZED

This situation led to the decision in 1929, when the culverts had been in service for 14 years, to make a careful examination which would definitely determine the facts regarding their condition. The need for such a survey was further emphasized by the fact that in the reports of all previous inspections made by the bureau and, so far as is known, in those of other organizations, the condition of the metal was described in general terms and did not make it possible to make direct and adequate comparisons of the behavior of the different culverts. It was realized that the application, in the proposed survey, of a rating schedule of condition, similar in principle to those developed by the Highway Research Board and several State highway departments, would obviate this difficulty and make possible a direct comparison of culvert behavior.

The rating of the condition of the metal in culverts fabricated from corrugated sheets has been developed to the point where, with the use of a suitable rating schedule, fairly accurate and comparable results may be expected. However, ratings depend on visual inspection and on the identification of certain rather definite stages in the life of the metal. The results, therefore, are subject to personal errors of observation and to a certain extent are influenced by personal opinion.

COMMITTEE OF 10 ORGANIZED TO CONDUCT INSPECTION AND SURVEY

This situation made it apparent that the accuracy of any report or conclusions based on the observations of an individual might be questioned. On the other hand, a report based on the findings of a number of observers, provided their ratings showed a reasonable degree of uniformity, would not be open to the suspicion of personal error or personal bias. It was decided to organize a group of representative engineers who would cooperate in the inspection and rating of the culverts and whose observations would be the basis of a report on the present condition of the various metals involved, with the exception of Mr. P. H. Everett, who died on May 24, 1930.

A group of 10 observers[2] was organized and the inspection on which this report is based was made in November, 1929.[3] The report has been reviewed and approved by each of these observers.

PROJECT DESCRIBED

The metal culverts in this project were installed during the spring and summer of 1915 on the Post Road between Austin and San Antonio. They were distributed over a distance of approximately 34 miles from a point about 4 miles north of San Marcos (1 mile south of the Blanco River) to a point about 12 miles south of New Braunfels. The general direction of the road is from northeast to southwest with numerous sections which do not follow this general direction. For the purpose of this report the road is considered to run from north to south, the directions to the right and left, when facing toward Austin, being considered as east and west, respectively.

Originally the installation included 3 galvanized pipes and 3 black (ungalvanized) pipes of each of 5 different metals making a total of 30 pipes. All pipes were 24 inches in diameter and of 14-gage (U. S. Standard) metal. These pipes were installed in 19 culvert structures, each having from one to four lines

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[1] U. S. Department of Agriculture Bulletin No. 586, pp. 27-30.

[2] The engineers who cooperated in making this survey and who, with the exception of Mr. Everett, have reviewed and approved this report, are as follows: R. W. Crum, director, Highway Research Board; J. B. Early, maintenance engineer, State Highway Department of Texas; G. G. Edwards, construction engineer, State Highway Department of Texas; J. D. Waldrop, State highway engineer, North Carolina State Highway Commission; S. B. Slack, bridge engineer, State Highway Board of Georgia; D. D. McGuire, engineer of tests, Tennessee Department of Highways and Public Works (since Apr. 1, 1930, division engineer, National Sand and Gravel Association, St. Louis); H. S. Gillette, materials engineer, district 6, Bureau of Public Roads; C. T. Nitteberg, highway bridge engineer, district 6, Bureau of Public Roads; the late P. H. Everett, senior highway engineer, district 6, Bureau of Public Roads; E. F. Kelley, chief, division of tests, Bureau of Public Roads.

[3] Acknowledgment is made to the State Highway Department of Texas for it, cooperation, through Mr. Gibb Gilchrist, State highway engineer, and his associates in placing the culverts in a condition suitable for inspection. Many of them were partially buried and a large amount of excavation was necessary to expose the ends sufficiently for inspection. Heavy rains just prior to the inspection also made a considerable amount of pumping necessary in order to expose the inverts of low culverts. This work was largely done prior to the arrival of the inspection party and permitted the observations to be made with a minimum expenditure of time and energy. Without this cooperation it would have been impossible to make the inspection in the time available.

TABLE 1.—*General information regarding culvert installation*

Culvert number or landmark	Station number [1]	Approximate total distance (odometer)	Number lines pipe in culvert structure	Designation of culvert pipe	Type of metal	Location of pipe in culvert	Direction of flow
HAYS COUNTY							
		Miles					
Blanco River, South Bank	1357	0					
No. 62	1419	1. 2	1	62	Black Bessemer steel		East.
No. 65	1457+50	1. 9	1	65	Galvanized Bessemer steel		Do.
No. 68	1515	3. 0	1	68	do		West.
North city limits San Marcos (San Marcos River)	1584	4. 3					
No. 74	1748+70	7. 5	1	74	Galvanized open-hearth steel		East.
No. 75	1765+35	8. 0	1	75	Black Bessemer steel		Do.
Hays-Comal County line:							
Hays County survey	1986	12. 4					
Comal County survey	539+16	12. 4					
COMAL COUNTY							
No. 94—I. & G. N. R. R. crossing	508	12. 9	1	94	Galvanized copper-bearing steel		Do.
No. 97	476+10	13. 5	1	97	Black copper-bearing iron		West.
No. 98	466+60	13. 7	1	98	Black copper-bearing steel		Do.
No. 101	393+85	15. 2	1	101	Galvanized copper-bearing steel		Do.
No. 103	355	15. 9	1	103	Black ingot iron		East.
No. 110	112+50	20. 6	2	110-N	Galvanized open-hearth steel	North pipe	Do.
				110-S	Black open-hearth steel	South pipe	Do.
North city limits, New Braunfels—Guadalupe River	0+00	23. 0					
South city limits, New Braunfels	539+16						
No. 127	716	29. 1	2	127-N	Galvanized ingot iron	North pipe	West.
				127-S	Galvanized copper-bearing iron	South pipe	Do.
No. 128	726	29. 3	2	128-N	Black copper-bearing iron	North pipe	Do.
				128-S	Galvanized copper-bearing iron	South pipe	Do.
No. 131	777	30. 3	2	131-N	Black open-hearth steel	North pipe	Do.
				131-S	Black copper-bearing steel	South pipe	Do.
No. 134	815	31. 0	1	134	Galvanized ingot iron		Do.
No. 143	937	33. 4	4	143-N	Galvanized copper-bearing steel	North pipe	Do.
				143-NC	Galvanized copper-bearing iron	North center pipe.	Do.
				143-SC	Black copper-hearing iron	South center pipe.	Do.
				143-S	Black copper-bearing steel	South pipe	Do.
No. 148	1012	34. 9	1	148	Black ingot iron		East.
No. 149	1018	35. 0	2	149-N	Galvanized ingot iron	North pipe	Do.
				149-S	Black ingot iron	South pipe	Do.
Comal-Guadalupe County line	1045+27						

[1] In Hays County the station numbers increase southward from station 1357 at the Blanco River to station 1986 at the Hays-Comal county line. In Comal County the station numbers decrease southward from station 539+16 (539+16=station 1986, Hays County survey), at the Hays-Comal county line to station 0+00 at the north city limits of New Braunfels, and increase southward from station 539+16 at the south city limits of New Braunfels to station 1045+27 at the Comal-Guadalupe county line.

of pipe. One culvert structure (No. 109) consisting of four lines of pipe, two galvanized and two black of each of two metals, was removed from the road prior to 1920. This reduced the number of pipes available for inspection to 26, and in the case of two metals reduced the number of both black and galvanized pipes from 3 to 2. The previous report states that all structures were provided with concrete head walls. At the present time several, toward the south end of the project, are provided with head walls of rubble masonry. The number, location, and general description of the existing culverts are given in Table 1.

DRAINAGE AND RAINFALL DISCUSSED

The general topography of this section of Texas is rolling. The drainage areas of the individual culverts range from flat to rolling, and, in the case of one or two culverts, to rather hilly. In several cases the drainage areas are so nearly level that, in the absence of flowing water, it is difficult to determine the direction of flow.

As regards rainfall, the official records of the stations of the United States Weather Bureau at San Marcos and New Braunfels may be considered representative since these towns, which are about 18 miles apart are both located within the project. The total annual rainfall and the average monthly rainfall at both of these stations for the period from 1915 to 1928, inclusive, are shown graphically in Figure 1. During this period the mean annual precipitation at New Braunfels was 30.3 inches and at San Marcos 32.4 inches. The minimum average monthly rainfall for the period was in excess of 1.5 inches at both stations. However, as would be expected, the records for individual years show a wide variation from the average.

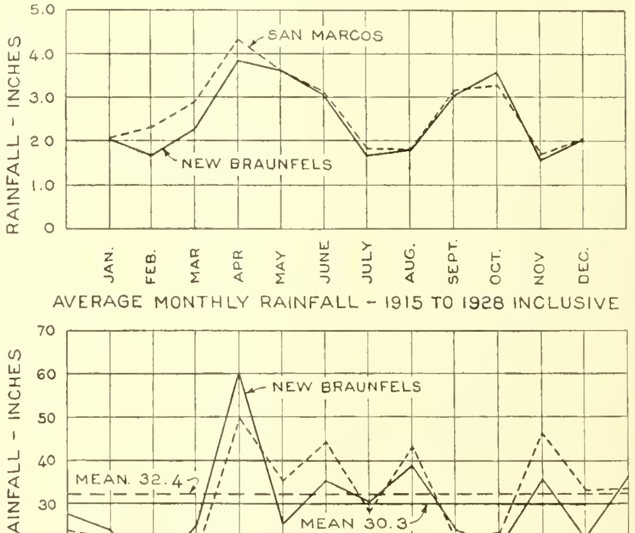

FIGURE 1.—GRAPHS SHOWING AVERAGE MONTHLY RAINFALL AND TOTAL ANNUAL RAINFALL FOR SAN MARCOS AND NEW BRAUNFELS, TEXAS

TABLE 2.—*Chemical characteristics of soils*

Sample No.	Sample taken at culvert number	Color of dry sample	Analysis of H_2O solution [1] (parts per million)				[2] Ph value	Description of sample
			CO_2	$CaCO_3$	Organic matter	Total soluble		
20	62	Gray white	1.1	120	40	161.1	8.8	Natural soil at west (inlet) end. Contained 93 per cent $CaCO_3$ and 6 per cent clay.
19	65	Brown gray	7.9	261	363	631.9	7.8	Filled material from inside of pipe. Contained much vegetable matter.
18	68	Dark gray	3.4	127	73	203.4	8.6	Natural soil at west (outlet) end.
17	74	Light gray	5.6	190	122	317.6	8.2	Natural soil at east (outlet) end.
16	94	Gray brown	4.5	125	155	284.5	8.4	Washed material from west (inlet) end.
15	94	Chocolate brown	4.5	175	145	324.5	8.2	Natural soil at west (inlet) end; rich in humus.
14	97	Yellowish brown	3.4	184	96	283.4	8.4	Natural soil at east (inlet) end; contained 70 per cent sand and gravel and 30 per cent clay.
13	101	Dark gray	3.4	123	125	251.4	8.6	Natural soil at west (outlet) end.
10	101	Light gray	2.3	146	118	266.3	8.8	Material washed through culvert.
8	103	Dark gray	2.3	123	122	247.3	8.6	Natural soil from west (inlet) end, taken from top 1 foot of natural ground.
9	103	do	3.4	125	73	201.4	8.6	Natural soil from elevation of culvert invert about 2½ feet below ground surface.
7	110	do	2.3	136	62	200.3	8.6	Natural soil at west (inlet) end.
12	131	Light gray	3.4	190	170	363.4	8.6	Natural soil at east (inlet) end.
11	131	Gray black	4.5	154	126	284.5	8.6	Filled material from inside of pipe 131-N.
5	134	Gray brown	3.4	166	98	267.4	8.6	Natural soil at west (outlet) end.
6	134	Dark gray	4.5	146	118	268.5	8.6	Filled material from inside of pipe.
3	148	Light gray	4.5	202	158	364.5	8.2	Natural soil at east (outlet) end.
4	148	do	3.4	177	167	347.4	8.4	Filled material from inside of pipe.
2	149	do	5.6	175	143	323.6	8.6	Natural soil at east (outlet) end.
1	149	do	3.4	80	144	227.4	8.8	Filled material from inside of pipe 149-N.

[1] Analysis of liquid extraction obtained by thoroughly dispersing 10 grams of material in 200 cubic centimeters distilled water, and, after all suspended matter had settled out, filtering off clear liquid.

[2] Ph determinations made on 1 gram of material in 10 cubic centimeters distilled water by colorimetric method using cresol red indicator having Ph values from 7.2 to 8.8. Ph values indicate the degree of concentration of hydrogen ions in solution, the neutral point being 7. For Ph values above 7 the reaction is basic; for values below 7 it is acid.

SOME CULVERTS FILLING, OTHERS SELF-CLEANING

At the present time 13 of the culvert pipes are in various stages of filling. This filling, in several instances, has progressed to the point where the pipe is almost or wholly inoperative as a drainage structure. The condition, in general, is due to the location of the culvert invert below the general level of the existing flow line of the drainage area. In certain cases it seems apparent that this defect existed at the time of construction; in others it is possible that changing conditions since the time of installation are responsible for it. The remaining 13 culvert pipes are generally free from deposits of silt and may be classed as self-cleaning rather than scouring since there is little or no evidence of abrasive action. From the conditions existing at the time of the inspection, which was made immediately after rather heavy rains, it is evident that flow occurs in none of the culverts except following rains and therefore may be classed as intermittent.

An examination of the rainfall records shows that normally there is an appreciable precipitation in all months of the year. This fact, together with the observed conditions surrounding the individual culvert installations, leads to the conclusion that the earth within all the culverts which have filled is generally in a wet or damp condition. On the other hand, because of the intermittent character of the flow, it is probable that, except following rains, the earth surrounding all the culverts is generally in a fairly dry condition. Subsequently, in this report, the soil condition is considered as damp in the case of filling culverts and dry in the case of self-cleaning culverts.

SOIL SAMPLES ANALYZED

Visual inspection indicates that the soil within the project is generally of the type known locally as "black-waxy," a soil which, when wet, is black and highly plastic. The mother rock with which it is underlain is reported to be a white, chalky limestone, and, toward the north end of the project, considerable amounts of this limestone are present in the soil in the form of gravel of varying size.

In connection with the survey, soil samples were taken at the sites of 13 of the 18 culvert structures. The samples are believed to be representative of the soil throughout the length of the project.

Liquid extractions were made from each soil sample and these were analyzed [4] for acidity, dissolved salts and organic matter. The results are reported in Table 2. The acid present was considered to be essentially CO_2. Quantitative tests for dissolved salts yielded only calcium carbonate and bicarbonate in weighable quantities. The lime was determined in the usual manner and all reported as $CaCO_3$. Organic matter was determined by deducting the weight of $CaCO_3$ found in a given volume of the soil water from the total weight of residue obtained by evaporating an equal volume of the same water to dryness and heating for 15 minutes at 110° C.

An outstanding characteristic of these soil waters is their extreme alkalinity, as indicated by the Ph values. It appears that nearly all the waters had about reached the point of saturation with regard to lime carbonate and consequently that the small quantity of free acid present might be expected to have a negligible corrosive effect on metal. Some corrosive action due to organic content might be anticipated, especially in the cases where appreciable quantities are indicated, but the high lime content should also mitigate against trouble from this source. That this is actually the case is indicated by the absence of any clear relationship, as will be shown later, between organic content and degree of corrosion. All the soil waters, with the exception of samples 19 and 20, were found to be very similar in regard to the quantity and quality of dissolved material. Sample 19 is exceptional on account of excess vegetable matter in the soil and sample 20 differs in being low in organic matter and acidity, owing to the high lime content.

CHECK ANALYSES OF CULVERT METALS MADE

At the time the culverts were installed, in 1915, samples of the metal were analyzed and the chemical analyses were presented in the previous published report. During the course of this survey check samples of the metal in each culvert pipe were taken and these were analyzed,[5] as a check on the original analyses and to insure the absence of error in the classification of the various metals. These check analyses are reported in Tables 3 and 4. As would be expected, there are some differences between the original analyses and the check analyses, the principal ones being in the carbon contents. These differences may be accounted for, at least partially, by the fact that in both series of analyses the amount of carbon was estimated by color comparison with standard samples. However, the agreement is sufficient for all practical purposes, since the slight discrepancies which exist are not sufficient to change the classification of the metal in any culvert pipe.

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[4] Analysis by Dr. E. C. E. Lord, associate petrographer, U. S. Bureau of Public Roads.

Table 3.—*Analyses of base metals of black (ungalvanized) culverts on the Austin-San Antonio Post Road. (Check samples)*[1]

Base metal	Culvert pipe number	Analysis of base metal							End of pipe sampled
		Carbon	Sulphur	Phosphorus	Manganese	Silicon	Total of the 5 elements	Copper	
		P. ct.	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	
Bessemer steel	62	0.055	0.048	0.096	0.360	0.002	0.561	Trace.	East.
Do	75	.050	.051	.100	.371	.005	.577	0.005	West.
Open-hearth steel	110-S	.045	.041	.016	.378	.006	.486	.005	East.
Do	131-N	.040	.041	.014	.385	.006	.486	.005	Do.
Copper-bearing steel.	98	.040	.036	.009	.434	.008	.527	.220	West.
Do	131-S	.040	.039	.010	.418	.008	.515	.220	East.
Do	143-S	.040	.043	.010	.434	.006	.533	.210	West.
Copper-bearing iron.	97	.015	.040	.009	.056	.008	.128	.140	Do.
Do	128-N	.020	.038	.009	.053	.006	.126	.140	Do.
Do	143-SC	.020	.041	.010	.056	.006	.133	.160	Do.
Ingot-iron	103	.010	.032	.006	.033	.004	.085	.005	East.
Do	148	.015	.032	.008	.037	Trace.	.092	.006	West.
Do	149-S	.020	.033	.008	.031	.005	.097	.006	Do.

[1] Analyses made of one sample from each culvert pipe.

Table 4.—*Analyses of base metals of galvanized culverts on the Austin-San Antonio Post Road. (Check samples)*[1]

Base metal	Culvert pipe number	Analysis of base metal							End of pipe sampled
		Carbon	Sulphur	Phosphorus	Manganese	Silicon	Total of the 5 elements	Copper	
		P. ct.	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	*P. ct.*	
Bessemer steel	65	0.050	0.050	0.102	0.350	0.004	0.556	0.004	West.
Do	68	.050	.056	.102	.357	.006	.571	.004	East.
Open-hearth steel	74	.045	.039	.024	.396	.004	.508	.005	Do.
Do	110-N	.040	.052	.016	.417	.004	.529	.005	Do.
Copper-bearing steel.	94	.040	.040	.009	.408	.006	.503	.195	West
Do	101	.035	.044	.011	.406	.007	.503	.180	East.
Do	143-N	.035	.042	.011	.411	.007	.506	.205	Do
Copper-bearing iron.	127-S	.015	.037	.010	.058	.006	.126	.120	Do.
Do	128-S	.025	.036	.009	.053	.004	.127	.140	Do.
Do	143-NC	.025	.039	.008	.051	.004	.127	.180	Do.
Ingot iron	127-N	.020	.041	.008	.018	.003	.090	.050	Do.
Do	134	.020	.042	.008	.028	Trace.	.098	.060	Do.
Do	149-N	.015	.042	.008	.018	Trace.	.083	.055	West.

[1] Analyses made of one sample from each culvert.

The five grades of sheet metal used in the fabrication of these corrugated pipe culverts were designated in the previous published report as Bessemer steel, open-hearth steel, copper-bearing steel, copper-bearing iron, and ingot iron.

The present standard requirements of the American Association of State highway officials for the base metals for corrugated metal pipe culverts are given in Table 5. These requirements are generally recognized by both sheet metal manufacturers and consuming interests as representing the essential chemical characteristics of the five grades of metal now considered suitable for use in metal culverts. It will be observed that Bessemer steel, and open-hearth steel without copper, are no longer used. Of the remaining metals used on the Austin-San Antonio Post Road, those designated as ingot iron and copper-bearing steel correspond, respectively, to the "pure iron" and "copper steel" of the association specification. The third, designated as copper-bearing iron, except for its deficiency in copper, corresponds closely to the "copper iron" of the association specification, although if account is taken of the tolerances permitted in the analyses of finished sheets, it might be considered to fall within the grade of "copper-bearing pure iron." However, the original designations are believed to be sufficiently descriptive of the grades of metal and are used throughout this report.

Table 5.—*Standard specifications for base metals for corrugated metal pipe culverts adopted by the American Association of State Highway Officials*[1]

Corrugated metal pipe culverts shall be fabricated from corrugated galvanized sheets, the base metal of which shall be made by the open-hearth process.

The base metal shall conform to the following chemical requirements:

Elements	Chemical composition by ladle analysis. Position of base metals does not indicate preference					Tolerance by check analysis of finished sheets
	Pure iron	Copper-bearing pure iron	Copper iron	Copper molybdenum iron	Copper steel	
Carbon, maximum per cent	------	------	------	------	------	------
Manganese, maximum per cent	------	------	------	------	------	------
Phosphorus, maximum per cent	0.015	0.015	0.015	0.015	------	------
Sulphur, maximum per cent	.040	.040	.040	.040	0.050	0.010
Silicon, maximum per cent	------	------	------	------	------	------
Copper, minimum per cent	------	.20	.20	.40	.20	.02
Molybdenum, minimum per cent	------	------	------	.05	------	------
Sum of first 5 elements, maximum per cent	------	.10	.25	.25	.70	.04
Sum of first 6 elements, maximum per cent	.10	------	------	------	------	.04

[1] Department Circular 331, United States Department of Agriculture, "Standard Specifications for Corrugated Metal Pipe Culverts Adopted by the American Association of State Highway Officials and Approved by the Secretary of Agriculture for Use in Connection with Federal-aid Road Work," November, 1925; Article 1, revised June 1, 1928.

CULVERTS INSPECTED AND RATED

The inspection of the culverts by the 10 observers was made on two successive days early in November, 1929. The excavations which were required at the ends of a number of the filling culverts in order to uncover them sufficiently for inspection resulted in the formation of sumps at the ends of the structures and the heavy rains immediately preceding the inspection filled these with water. This resulted in some inconvenience, as it was necessary to remove the water with hand pumps, but it also proved to be a distinct benefit in that it made available the means for thoroughly cleansing the metal surfaces prior to inspection.

As previously mentioned, 13 of the 26 culvert pipes are in various stages of filling. On this account it was not practical to inspect the pipes for their full length. The condition of the metal, for both filling and self-cleaning culverts, was determined by inspection of from 2 to 3 feet of the interior at each end of the pipe.

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[5] Chemical analyses by L. G. Carmick, associate chemist, Bureau of Public Roads.

Each observer was supplied with blank forms upon which to record his observations and a rating schedule to be used in designating the various stages of deterioration of the metal.

This rating schedule, suggested by Mr. R. W. Crum, was as follows:

Rating schedule

	Rating
SPELTER STAGE	
Spelter intact and spangles[6] clearly defined	90
Spelter dark and appreciably soft, spangles obliterated	75
Spelter practically gone, thin rust forming in spots, no pitting	70
METAL STAGE	
Uniform rust coat, extremely small pits	65
Uniform rust, deeper pits. Light warty or nodular growth	60
Deep pits, considerable loss of metal	50
Pits about half through	35
Heavy warty and nodular growth, metal very thin	25
Base metal perforated on invert	10
Base metal practically gone on invert	0

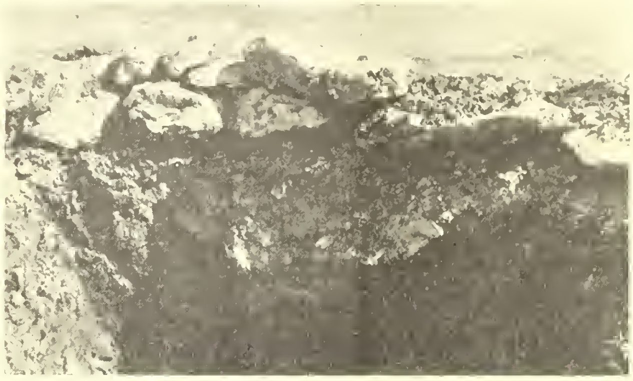
It has been customary, with rating schedules of this type, to assume a straight-line deterioration of the metal and to consider the rating of a given metal condition, on the basis of a rating of 100 for perfect condition, as indicative of a definite point in the life of the structure. In contrast to this practice the numerical ratings used in this survey were considered as indices or code designations of the various stages in deterioration which can be identified with some degree of precision. They were of practical advantage in permitting the observations of the different observers to be averaged, but, while the ratings are arranged in the descending numerical order for the advancing stages of corrosion, the numerical ratings themselves should not be considered as indicating definite points in the life of the structures rated or as giving an accurate indication of the rate of deterioration. Therefore, the ratings which are reported give no adequate basis for estimating the probable future life of the structures.

"BLACK" AND GALVANIZED GROUPS CONSIDERED INDEPENDENTLY

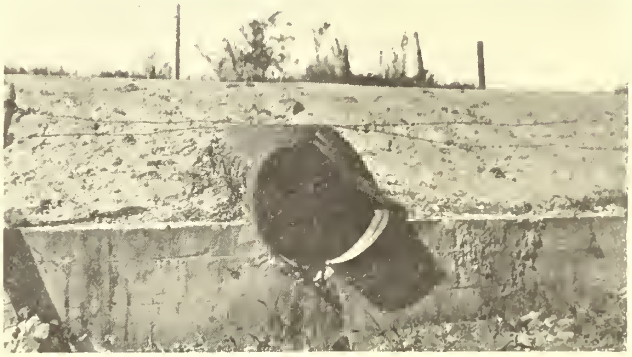
The rating schedule was based on a large amount of previous experience obtained in the examination of culverts fabricated from galvanized sheets. Since black (ungalvanized) sheets are not used in the manufacture of culverts of this type, there existed no information on which to base a schedule for use in rating the black culverts. Due to this fact, the same schedule was used in rating both the galvanized and black culverts with the result that the latter were always rated as being in the "metal stage." Therefore, in the ratings which are reported the maximum possible rating for a galvanized culvert is 90, while the maximum possible rating for a black culvert is 65. Because of the difference in the basis of rating and the fact that, when installed, the galvanized culverts had the advantage of a protective coating of zinc spelter, there exists no means for making direct comparisons of the behavior of the same metal in the two classes of culverts. Accordingly, the two classes, black and galvanized, are considered as independent groups.

TECHNIQUE OF INSPECTION ASSURED IMPARTIAL RESULTS

In making the inspection the observers, subsequently identified by the letters A, B, C, D, E, F, G, H, I, and J, traveled in a group. Each individual made such observations as he thought necessary, and, entirely independently of the others, reached his own conclusions as to the proper ratings to be applied and entered them on his record sheets. There was little or no discussion during the inspection of the ratings which were being given, and it was very apparent that the decisions of each were actually made independently of all others in the party. This independence of action is evidenced by the actual ratings which were reported.



INLET END (WEST)



OUTLET END (EAST)

Figure 2.—Culvert No. 103. Inlet end completely filled with debris, while outlet is unobstructed and entirely free from filling material

In order to avoid all suspicion of personal bias, the observers were not supplied with information relative to the grades of metal in the various culverts until after the ratings had been made. While this information had been public property for a number of years, it is believed that none of the observers had made an effort to become familiar with it and, with few exceptions, the culverts were rated without knowledge of the grade of metal which was being observed.

During the first day of the survey all observers inspected and rated all the culverts on the project. A second and more hurried inspection was made on the following day to permit a check of the first observations and any adjustment of the original ratings which might be thought necessary. The second inspection was made by observers A to F, inclusive, observers G, H, I, and J not being present. The light was better on the second day than on the first and made it possible for the surface condition of the metal to be seen somewhat more distinctly. Adjustments were made in some of the original ratings as a result of this second inspection. It may be said, therefore, that the final ratings as reported by observers A to F, inclusive, are based on a more extended inspection, a portion of which was made under more favorable conditions, than those of observers G to J, inclusive. However, it can not be said that the ratings of the former group are necessarily any more accurately indicative of the actual conditions.

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[6] "Spangles" are the crystalline patterns in the zinc spelter, characteristic of new galvanized coatings.

The inlet and outlet ends of each pipe were rated, except in the cases of Nos. 94 and 103, where only one end was accessible for inspection. The ratings of the two ends were averaged to obtain the rating of the pipe. The final rating of all observers for all pipes, together with the average of the ratings of all observers for each pipe, are given in Tables 6 and 7.

TABLE 6.—*Ratings of black (ungalvanized) culverts on the Austin-San Antonio Post Road*

Pipe No.	62			75			97			98			103			110-S			128-N			131-N			131-S			143-SC			143-S			148			149-S		
Observer	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average
A	35	10	22½	10	35	22½	60	60	60	60	60	60	--	25	25	10	10	10	60	50	55	60	50	55	60	50	55	65	65	65	65	65	65	60	60	60	60	60	60
B	25	10	17½	10	35	22½	50	60	55	60	50	55	--	10	10	10	10	10	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	60	65	62½
C	25	10	17½	10	35	22½	60	60	60	60	60	60	--	25	25	10	10	10	60	60	60	50	35	42½	50	50	50	65	65	65	65	60	62½	60	65	62½	65	65	65
D	25	10	17½	10	25	17½	25	25	25	25	25	25	--	25	25	10	10	10	50	50	50	60	50	55	60	50	55	60	60	60	60	60	60	60	60	60	60	65	62½
E	25	10	17½	10	35	22½	35	60	47½	60	50	55	--	10	10	10	10	10	60	60	60	60	50	55	65	50	57½	65	60	62½	60	60	60	60	65	62½	60	65	62½
F	25	10	17½	10	50	30	50	60	55	65	60	62½	--	25	25	10	10	10	50	50	50	65	60	62½	65	60	62½	65	60	62½	65	60	62½	65	65	65	60	65	62½
G	35	35	35	10	25	17½	60	60	60	60	60	60	--	10	10	10	10	10	60	60	60	65	60	62½	65	60	62½	60	60	60	60	60	60	60	65	62½	60	60	60
H	35	10	22½	10	25	17½	60	60	60	60	60	60	--	10	10	0	0	0	60	60	60	50	50	50	50	50	50	65	65	65	65	65	65	60	60	60	60	60	60
I	50	10	30	10	50	30	60	60	60	65	65	65	--	35	35	10	10	10	60	60	60	65	65	65	65	65	65	60	50	55	60	50	55	65	65	65	65	65	65
J	50	35	42½	50	60	55	65	65	65	60	60	60	--	35	35	10	10	10	60	50	55	60	60	60	60	60	60	60	50	55	60	50	55	60	65	62½	60	65	62½
Average	33	15	24	14	37½	25¾	52½	57	54¾	57½	55	56¼	--	21	21	9	9	9	58	56	57	59½	54	56¾	60	55½	57¾	62½	59½	61	62	59	60½	61	63	62	61	63½	62¼

TABLE 7.—*Ratings of galvanized culverts on the Austin-San Antonio Post Road*

Pipe No.	65			68			74			94			101			110-N			127-N			127-S			128-S			134			143-N			143-NC			149-N		
Observer	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average	Inlet	Outlet	Average
A	90	90	90	65	70	67½	90	90	90	90	--	90	65	65	65	90	----	90	70	65	67½	70	65	67½	65	60	62½	70	70	70	75	75	75	70	75	72½	65	65	65
B	90	75	82½	60	70	65	90	90	90	90	--	90	65	60	62½	75	90	82½	70	65	67½	65	65	65	65	60	62½	70	70	70	90	75	82½	70	70	70	65	65	65
C	90	90	90	65	70	67½	90	90	90	90	--	90	65	65	65	90	90	90	70	65	67½	70	65	67½	70	70	70	70	70	70	90	90	90	75	75	75	70	70	70
D	90	90	90	60	70	65	90	90	90	90	--	90	65	60	62½	90	90	90	70	70	70	70	65	67½	70	65	67½	70	70	70	90	75	82½	70	75	72½	65	60	62½
E	90	90	90	75	70	72½	90	90	90	90	--	90	65	65	65	90	90	90	70	65	67½	70	65	67½	65	70	67½	70	70	70	90	75	82½	70	75	72½	60	70	65
F	90	90	90	65	70	67½	90	90	90	90	--	90	65	60	62½	75	90	82½	70	70	70	70	60	65	65	65	65	70	70	70	90	90	90	65	70	67½	60	65	62½
G	75	75	75	75	70	72½	75	75	75	75	--	75	70	65	67½	90	90	90	70	70	70	75	75	75	70	70	70	75	75	75	75	75	75	70	70	70	70	70	70
H	90	90	90	70	70	70	75	75	75	90	--	90	70	70	70	90	90	90	75	75	75	75	75	75	70	90	80	75	75	75	90	90	90	75	75	75	70	70	70
I	75	75	75	60	70	65	90	90	90	90	--	90	70	70	70	90	90	90	75	75	75	75	75	75	75	75	75	90	90	90	90	90	90	75	75	75	65	70	67½
J	75	75	75	70	75	72½	75	75	75	75	--	75	70	70	70	90	75	82½	70	70	70	70	70	70	75	70	72½	75	75	75	75	75	75	75	75	75	70	70	70
Average	85½	84	84¾	66½	70½	68½	85½	85½	85½	87	--	87	67	65	66	87	88⅓	87¾	71	69	70	71	68	69½	69	69½	69¼	73½	73½	73½	85½	81	83¼	71½	73½	72½	66	67½	66¾

RESULTS DISCUSSED AND ANALYZED

An examination of the tabulated ratings shows generally excellent agreement between the different observers. This is rather conclusive evidence that the general principles involved in this method of measuring the behavior in service of this class of drainage structure are sound, since it shows that comparable results may be expected from the use, by different individuals, of the same scheme of rating or classification. This evidence is further strengthened by the fact that the majority of the observers who established these ratings were relatively inexperienced in the inspection and rating of metal culverts. It is logical to assume that even more concordant results might be expected from the experienced men who normally would conduct any large-scale survey of this type.

In examining the ratings for consistency, certain characteristics of the rating schedule itself should be given consideration since these are such as to make the same difference between the ratings of two observers of more significance at one place in the scale than in another. For instance, in the spelter stage the rating may be 90 or 75 depending entirely on the presence or absence of spangles. Since the first stage tends to shade rather imperceptibly into the second, there frequently may be a question as to the actual obliteration of the spangles, and the decision depends not only on power of observation but also on opinion as to what constitutes a clearly defined spangle. For this reason the fact that one group of observers may give a culvert a rating of 90 while another group may give the same culvert a rating of 75, does not necessarily indicate inaccuracy on the part of either group. On the other hand, a rating of 10 indicates the very definite and obvious stage in which the metal is actually perforated, while a rating of 25 indicates a very thin condition of the metal but one in which holes have not yet actually developed. Ratings, by different observers, of 10 and 25 for the same culvert therefore indicate actual error of observation.

It is obvious, from these examples and others which might be mentioned, that certain stages of deterioration defined in the rating schedule may be identified with more precision than others. The ratings, therefore, differ from other data of an experimental nature in that they do not lend themselves to a mathematica analysis for uniformity. For instance, figures showing mean deviations from average results are practically meaningless, since they give no adequate conception o the differences which may be significant and those which may not. It is believed that an appreciation o the degree of uniformity of the ratings can be obtained only by visual inspection of the tables, having in mind the charater of the rating schedule.

UNGALVANIZED CULVERTS SHOW SOME INCONSISTENCIES IN RATING

For the black culverts there are noticeable differences in the ratings of pipes Nos. 62, 75, 97, 98, and 103. As compared with ratings of the other observers, the ratings of observers G, I, and J are high for No. 62; that of observer J is high for No. 75; those of observers D and E are low for No. 97; that of observer D is low for No. 98; and those of observers I and J are high for No. 103. The differences are attributed largely to these observers failing to see conditions which were obvious to the majority, or detecting conditions which were unobserved by the other inspectors. In the cases of pipes Nos. 97 and 98 they may be due to a difference in the interpretation of conditions which were probably obvious to all. However, the elimination of the ratings which vary widely from those of the majority would not result in any marked change in the average ratings for the group.

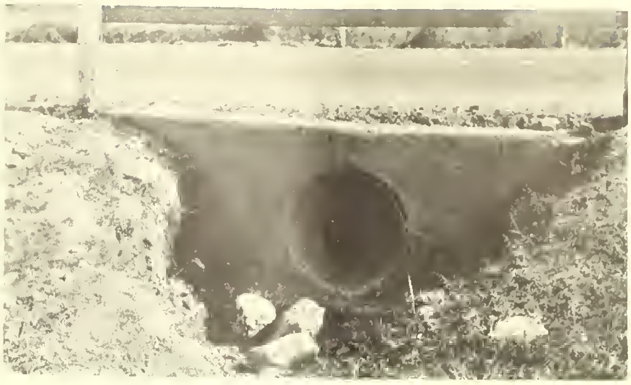
RATINGS OF GALVANIZED CULVERTS UNIFORMLY HIGH

The ratings of the galvanized culverts are much more consistent than those of the black culverts if the difference, already discussed, between the ratings of 90 and 75 is considered. In fact, there are no discrepancies of sufficient magnitude to warrant discussion.

The significant fact developed by the ratings of the galvanized pipes is their generally excellent condition after having been in service for more than 14 years. With the rating schedule used in this survey, 90 was the highest rating which could be given a galvanized pipe, while a rating of 70 indicated that the metal was still in the spelter stage. Having this in mind, it is interesting to note that 6 of the 13 galvanized pipes were given an average rating of 90 by one or more observers; that the lowest average rating given any pipe by any observer was 62½, only 2½ points below the highest rating which could be given a black pipe; and that for only 5 pipes were the averages of the ratings of all observers below 70. The conclusion which may be drawn from this is that up to the time of this inspection the performance of the galvanized culverts has constituted a test of spelter coatings and not of base metals. Corrosion had not progressed to a point which would permit of any comparison of the corrosion-resisting properties of the base metals in the galvanized pipes. Any comparison which may be made of the relative behavior of base metals in the culverts in this project must be based entirely on a consideration of the black pipes.

DATA NOT ADEQUATE TO GIVE DEFINITE CONCLUSIONS REGARDING BASE METALS

The major purpose of this experimental culvert installation was to determine the relative merits of the different grades of base metal which were used, and naturally considerable interest regarding this phase attaches to the findings of the present survey. Before undertaking any discussion of relative durability as indicated by the condition ratings, the inadequacy of the experiment, from the standpoint of evidence available, must be emphasized. If we consider only the three variables, grade of metal, protective coating and soil condition (damp and dry, for filling, and self-cleaning culverts, respectively), then there are 20 possible combinations of these variables, with only 26 culvert pipes in the project. In Table 8 the number of pipes for each of these 20 combinations of variables is given. It will be seen that there are four conditions for which there is no evidence whatever and that in no case are there more than three pipes for any one condition. Statistical studies of a large amount of data obtained in culvert condition surveys of this type have indicated that a considerable number of samples, possibly between 50 and 100 for each condition, must be available for study if results having any reasonable degree of accuracy are to be expected.


INLET END (WEST)

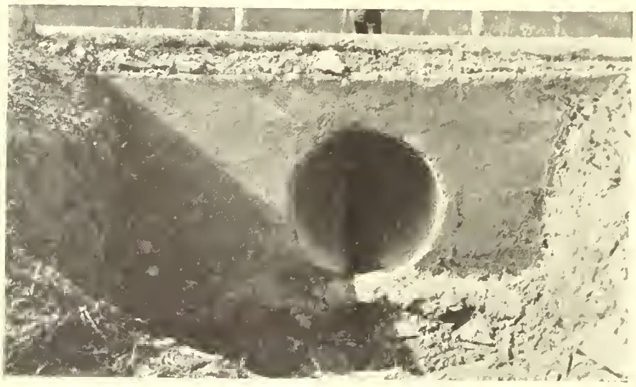

OUTLET END (EAST)

FIGURE 3.—CULVERT NO. 74. A TYPICAL SELF-CLEANING CULVERT

TABLE 8.—*Number of culvert pipes for each combination of variables*

Base metal	Black or galvanized	Number of culvert pipe		
		Total	Filling, soil condition generally damp	Self-cleaning, soil condition generally dry
Bessemer steel	Black	2	0	2
	Galvanized	2	2	0
Open-hearth steel	Black	2	1	1
	Galvanized	2	0	2
Copper-bearing steel	Black	3	1	2
	Galvanized	3	1	2
Copper-bearing iron	Black	3	1	2
	Galvanized	3	2	1
Ingot iron	Black	3	2	[1] 1
	Galvanized	3	3	0
Total		26	13	13

[1] Culvert pipe No. 103. See explanation of this classification in body of report.

The following discussion, therefore, concerns only the relative behavior of the limited number of pipe in this particular project. To assume either that the same trend of results would have been observed had a greater number of pipe been available for inspection, or that the same differences in behavior of the various metals would obtain in other localities, would be unwarranted.

TABLE 9.—*Summary of average ratings—ungalvanized pipe*

Culvert pipe No.	Average rating [1]	Base metal	Filling or self-cleaning	Organic content of soil [2]
				Parts per million
149-S	62¼	Ingot iron	Filling	144
148	62	do	do	158-167
143-SC	61	Copper-bearing iron	Self-cleaning	
143-S	60½	Copper-bearing steel	do	
131-S	57¾	do	Filling	126-170
128-N	57	Copper-bearing iron	do	
131-N	56¾	Open-hearth steel	do	126-170
98	56¼	Copper-bearing steel	Self-cleaning	
97	54¾	Copper-bearing iron	do	96
75	25¾	Bessemer steel	do	
62	24	do	do	40
103	21	Ingot iron	do.[3]	73-122
110-S	9	Open-hearth steel	do	62

[1] From Table 6.
[2] From Table 2.
[3] See explanation of this classification in body of report.

TABLE 10.—*Summary of average ratings—galvanized pipe*

Culvert pipe No.	Average rating [1]	Base metal	Spelter coating [2]	Filling or self-cleaning	Organic content of soil [3]
			Oz. per sq. ft.		*Parts per million*
110-N	87¾	Open-hearth steel	2.58	Self-cleaning	62
94	87	Copper-bearing steel	2.48	do	145-155
74	85½	Open-hearth steel	2.84	do	122
65	84¾	Bessemer steel	2.59	Filling	363
143-N	83¼	Copper-bearing steel	2.48	Self-cleaning	
134	73½	Ingot iron	2.41	Filling	98-118
143-NC	72½	Copper-bearing iron	2.02	Self-cleaning	
127-N	70	Ingot iron	2.44	Filling	
127-S	69½	Copper-bearing iron	1.90	do	
128-S	69¼	do	1.86	do	
68	68½	Bessemer steel	2.80	do	73
149-N	66¾	Ingot iron	2.50	do	144
101	66	Copper-bearing steel	2.59	do	118-125

[1] From Table 7. [2] From U. S. D. A. Bulletin No. 586. [3] From Table 2.

In Tables 9 and 10 are summarized the average ratings for black and galvanized pipes, respectively, the arrangement of the different pipes being in the descending order of excellence from the corrosion standpoint. In these tables are included the results, from Table 2, of the organic content analyses of the soils at the various locations. It is evident, as has been previously stated, that there is no clear relationship between organic content and degree of corrosion.

Table 9 shows that culvert pipe No. 103 has been classified as in a self-cleaning condition. The observers, without exception, in reporting their field observations, classified this pipe as in a filling condition, and it is believed that this classification was based entirely on the condition of the inlet end of the pipe which was so completely filled with débris that an inspection of this end could not be made. The rating of this pipe is based entirely on inspection of the outlet end which was unobstructed and contained no filling material. The condition of the two ends at the time of inspection is shown in Figure 2. The low rating of pipe No. 103, as compared with the high ratings of pipes Nos. 149-S and 148 of the same metal, suggests that a difference in service conditions may be at least partly responsible for the wide difference in behavior. It is believed that any comparisons may be made on a fairer basis if pipe No. 103 is classified as in the self-cleaning condition, which was the actual condition of the end which was rated, and for the purpose of this report this has been done.

In Table 11 are given, for the black culvert pipe, the averages of the average ratings for each base metal, the metals being arranged in the descending order of excellence. This table also shows the relative order of excellence as indicated by the average ratings of each observer. The generally good agreement between the trends of excellence shown by the ratings of the individuals and those shown by the average ratings of the whole group is worthy of note. The ratings of six observers show the same general order of excellence as is shown by the average ratings of the group and all observers are in agreement that the group of metals which includes copper-bearing steel, copper-bearing iron and ingot iron has proved superior in corrosion resistance to the group which includes open-hearth steel and Bessemer steel.

TABLE 11.—*Ungolvanized pipe. Average ratings on the basis of base metals, and relative order of excellence as indicated by these ratings and the average ratings of each observer*

Base metal	Average of all ratings	Relative order of excellence [1]—Observers					
		All	E, F, G, and I	A and B	C and H	D	J
Copper-bearing steel	58.2	1	1	1	2	2	1
Copper-bearing iron	57.6	2	2	1	1	3	1
Ingot iron	48.4	3	3	2	3	1	2
Open-hearth steel	32.9	4	4	3	4	4	4
Bessemer steel	24.9	5	5	4	5	5	3

[1] When the figures indicating order of excellence for any one observer are the same for more than one metal they signify that the average ratings of that observer are the same for all metals to which the same figure is assigned.

DATA ANALYZED BY GROUPS, ON THE BASIS OF SELF-CLEANING AND FILLING CONDITIONS

Visual inspection of Table 9 indicates a rather marked superiority for the black culverts in the filling condition while Table 10 indicates an equally marked superiority for the galvanized culverts in the self-cleaning condition. The average ratings for the black culverts are 59.2 for the filling condition and 39 for the self-cleaning condition; for the galvanized culverts the average ratings are 83.2 for the self-cleaning condition and 71 for the filling condition. These facts have suggested the presentation of the data in another form involving the separation of the culvert pipe into groups on the basis of self-cleaning and filling condition. This has been done in Tables 12 to 15, inclusive, which are of the same general form as Table 11.

Table 12, for black pipe in the self-cleaning condition, shows the same superiority of copper-bearing steel and copper-bearing iron that was indicated in Table 11. Ingot iron has lost its place among the first three metals on account of the poor condition of the one pipe, No. 103, included in this comparison. All observers are in agreement that, for the self-cleaning condition, copper-bearing steel and copper-bearing iron are superior to the other three metals and all are in agreement that the one open-hearth steel pipe has shown the least resistance to corrosion. However, as regards the relative positions of Bessemer steel and ingot iron, the ratings of only 5 observers of the 10 are in agreement with the average ratings of the group.

On the other hand, the same three metals which showed superiority in Table 11 also show superiority in Table 13, for black pipe in the filling condition, although in the latter table the order is changed, ingot iron occupying first position. The agreement between the trends of excellence shown by the ratings of the individuals and those shown by the average ratings of the whole group is not as good as that indicated in Tables 11 and 12 but this is not surprising in view of the small differences between the average ratings of the metals in Table 13.

The data available, as presented in Tables 11, 12, and 13, indicate that, for the black culverts under the

TABLE 12.—*Ungalvanized pipe—Self-cleaning condition. Average ratings on the basis of base metals, and relative order of excellence as indicated by these ratings and the average ratings of each observer*

Base metal	Number of pipe	Average of all ratings	Relative order of excellence [1]—Observers							
			All	A and D	B and G	F and I	C	E	H	J
Copper-bearing steel	2	58.4	1	1	1	1	2	1	1	2
Copper-bearing iron	2	57.9	2	1	1	2	1	2	1	1
Bessemer steel	2	24.9	3	3	2	4	4	3	2	3
Ingot iron	1	21	4	2	3	3	3	4	3	4
Open-hearth steel	1	9	5	4	3	5	5	4	4	5
Average		39.0								

[1] When the figures indicating order of excellence for any one observer are the same for more than one metal they signify that the average ratings of that observer are the same for all metals to which the same figure is assigned.

TABLE 13.—*Ungalvanized pipe—filling condition. Average ratings on the basis of base metals, and relative order of excellence as indicated by these ratings and the average ratings of each observer*

Base metal	Number of pipe	Average of all ratings	Relative order of excellence [1]—Observers						
			All	A and B	C and E	D, F, and J	G	H	I
Ingot iron	2	62.1	1	1	1	1	2	1	1
Copper-bearing steel	1	57.8	2	2	3	2	1	2	1
Copper-bearing iron	1	57	3	2	2	3	3	1	2
Open-hearth steel	1	56.8	4	2	4	2	1	2	1
Average		59.2							

[1] When the figures indicating order of excellence for any one observer are the same for more than one metal they signify that the average ratings of that observer are the same for all metals to which the same figure is assigned.

TABLE 14.—*Galvanized pipe—self-cleaning condition. Relative order of excellence as indicated by the average ratings of all observers and the ratings of each observer*

Culvert pipe No.	Average rating	Spelter coating	Relative order of excellence [1]—Observers							
			All	A, D, and E	C and I	B	F	G	H	J
		Oz. per sq. ft.								
110-N	87¾	2.58	1	1	1	2	2	1	1	1
94	87	2.48	2	1	1	1	1	2	1	2
74	85½	2.84	3	1	1	1	1	2	2	2
143-N	83¼	2.48	4	2	1	2	1	2	1	2
143-NC	72½	2.02	5	3	2	3	3	3	2	2
Average	83.2									

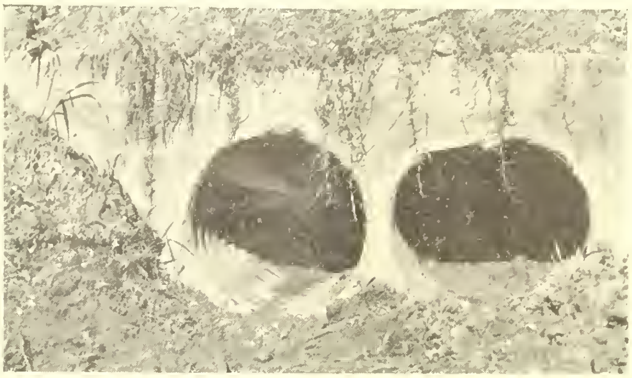
[1] When the figures indicating order of excellence for any one observer are the same for more than one culvert pipe they signify that all pipe to which that figure is assigned were given the same rating by the observer.

TABLE 15.—*Galvanized pipe—filling condition. Relative order of excellence as indicated by the average of the ratings of all observers and the ratings of each observer*

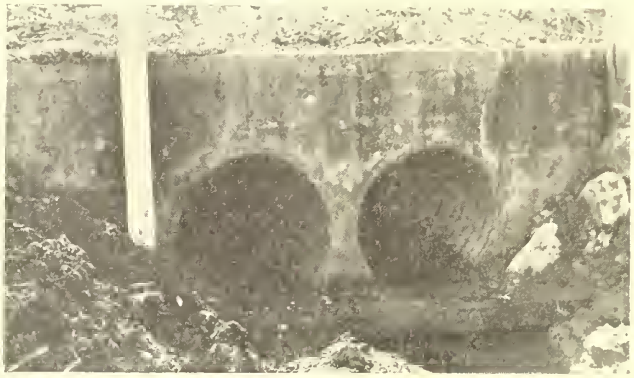
Culvert pipe No.	Average rating	Spelter coating	Relative order of excellence [1]—Observers										
			All	A	B	C	D	E	F	G	H	I	J
		Oz. per sq. ft.											
65	84¾	2.59	1	1	1	1	1	1	1	1	1	2	1
134	73½	2.41	2	2	2	2	2	3	2	1	3	1	1
127-N	70	2.44	3	3	3	3	2	4	2	3	3	2	3
127-S	69½	1.90	4	3	4	3	3	4	4	1	3	2	3
128-S	69¼	1.86	5	5	5	2	3	4	4	3	2	2	2
68	68½	2.80	6	3	4	3	4	2	3	2	4	5	2
149-N	66¾	2.50	7	4	4	2	5	5	5	3	4	4	3
101	66	2.59	8	4	5	4	5	5	5	4	4	3	3
Average	71.0												

[1] When the figures indicating order of excellence for any one observer are the same for more than one culvert pipe they signify that all pipe to which that figure is assigned were given the same rating by the observer.

conditions obtaining in this locality, the filling condition is more favorable to long life than is the self-cleaning condition, and that the group of metals which includes copper-bearing steel, copper-bearing iron, and ingot iron is superior, in resistance to corrosion, to the group which includes open-hearth steel and Bessemer steel.



INLET END (EAST)



OUTLET END (WEST)

FIGURE 4.—CULVERT NO. 128. A TYPICAL FILLING CULVERT

As has been pointed out previously, the ratings of the galvanized culverts give no indication of the corrosion resisting properties of the base metals. For this reason the average ratings of the individual pipe are given in Tables 14 and 15, for galvanized pipe in the self-cleaning and filling conditions, respectively, and the pipe have not been grouped with respect to base metals as was done in Tables 11 to 13, inclusive. It is apparent, therefore, that Tables 14 and 15 can disclose no new facts in addition to those already shown in Table 10, except a possible relationship between weight of spelter coat and resistance to corrosion, and the degree of agreement between the trends of excellence shown by the ratings of individual observers and those shown by the average ratings of the group.

It seems reasonable to assume that the weight of the spelter coating should have an influence on the behavior of galvanized pipe, the heavier coatings being more favorable to long life than the lighter ones. However, the data in Tables 14 and 15 show no consistent relationship between weight of spelter and culvert rating and permit no conclusion to be drawn.

The agreement between the trends of excellence shown by the ratings of the individual observers and those shown by average ratings of the whole group is not as good for the galvanized pipe as for the black. This is doubtless due to the smaller range in average

(Continued on page 184)

METHODS FOR THE MEASUREMENT OF WATER FOR CEMENT BRIQUET TESTS

TWO methods for the measurement of quantities of water used in mixing of cement mortar for the standard briquet test are described in the following articles. In each case a burette of special design is employed, and methods of filling the burette to a constant predetermined level are used. The amount of water drawn from the burette is, however, dependent on the operator and in this respect the measurements described are subject to variation. In addition, a fully automatic method of measuring water is suggested in one article.

METHOD USED IN THE MISSOURI STATE HIGHWAY LABORATORY

By F. V. Reagel, Engineer of Materials, Missouri State Highway Department

A simple method for measuring the water used in gauging mortar batches for briquet tests, which has been used in the laboratory of the Missouri State Highway Department for several years, has assisted in promoting the accuracy and speed of these tests. Primarily, the device is merely an adaptation of the automatic burette having an intake, an outlet, and an overflow of small diameter which drains into a sump. The intake is connected, through a brass turncock, to a large bottle of distilled water. A rubber tube, with a pinch cock at its lower end, extends from the outlet to the work table. The sump drains into a small overflow bottle. The burette bulb has a volume of 100 cubic centimeters from the upper graduation on the neck to the mouth of the overflow; the neck has a volume of 10 cubic centimeters with graduations every 0.5 cubic centimeter which are approximately 5 millimeters apart. This permits measurement of any quantity of water from 100 to 110 cubic centimeters to within 0.25 cubic centimeter.

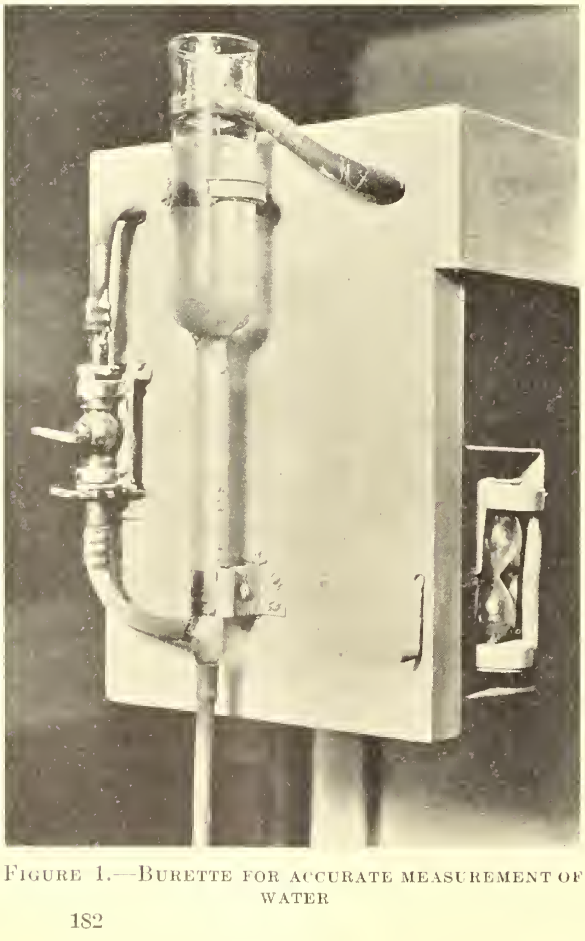


Figure 1.—Burette for accurate measurement of water

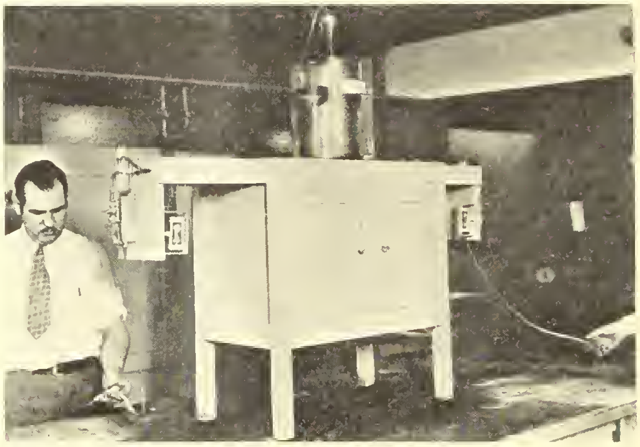


Figure 2.—Apparatus as installed in the Missouri laboratories

In operation, the intake cock is turned on until water overflows and is then shut off. The pinch cock is then opened and the proper amount of water, as indicated by the burette graduations, is permitted to flow into the dry cement-sand mixture. The intake cock is then immediately opened so that the burette will fill while the operator is mixing the batch. Much less time is consumed by this method than by the use of the ordinary graduated cylinder, as the filling is done automatically while the operator is doing something else, and the measurement is merely a matter of releasing the pinch cock when the water level reaches the proper graduation. Furthermore, the measurements are more accurate, as the burette is easily read to 0.25 cubic centimeter (about one-fourth of 1 per cent), whereas it is not possible to measure closer than 2 cubic centimeters (2 per cent) with a graduated cylinder. The burette was made to order at a relatively small cost.

The equipment as used in this laboratory is shown in Figures 1 and 2. This arrangement with a burette at each end of the rack accommodates four operators. The method of suspending the minute-glass is also a useful innovation, as it keeps the minute-glass out of the operator's way, on a level with and immediately in front of his eyes.

METHOD USED IN THE LABORATORY OF THE BUREAU OF PUBLIC ROADS

By D. O. WOOLF, Associate Materials Engineer, Bureau of Public Roads

The apparatus for measuring water for mortar which has been recently installed in the laboratory of the Bureau of Public Roads is an adaptation of the constant-level water bottle employed in connection with the steam chest for testing the soundness of cement. As shown in Figure 3, a burette of special design is connected by rubber tubing with a five gallon water bottle. This bottle is closed with a rubber stopper through which a length of ¼-inch glass tubing passes. The bottom of the tubing is placed in the same horizontal plane with the zero mark on the burette.

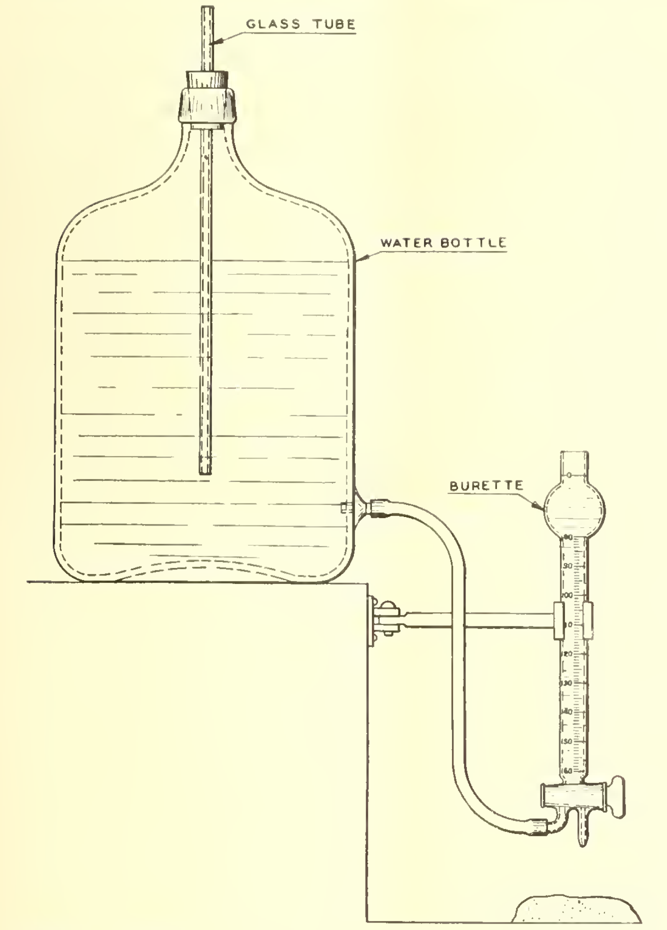


Figure 3.—Semi-automatic apparatus for measuring water

As the mouth of the water supply bottle is closed, the effective water level is at the bottom of the glass tube which enters the bottle. If water is permitted to flow from the bottle to the burette, the water level in the burette will rise until it reaches the same height as the bottom of the tube in the bottle.

In operation, either the burette or the glass tubing is adjusted so that the bottom of the tube is in the same horizontal plane with the zero mark on the burette. This may readily be accomplished by sliding the burette up or down in its support as is necessary, or by changing the height of the tube. The two-way stopcock on the burette is then opened to permit water to flow into the burette. After the latter is filled the required amount of water is drained out and the stopcock turned to the filling position. The water will again rise to the zero mark on the burette without attention of the operator. If the burette is drained completely, it will refill in about one minute.

The chief advantage of this installation is that it relieves the operator of making one of the two readings necessary for each quantity of water drawn from the burette. The apparatus is relatively simple, and in normal operation the results are accurate. The only feature which must be guarded against is loss of the partial vacuum in the supply bottle. If, after the burette has been adjusted to position, leakage of air occurs around the stopper of the supply bottle, the water level in the tube will rise due to the increase in pressure of the air entrapped in the bottle. This will permit a larger amount of water to enter the burette than is desired. It is necessary, therefore, to note that no water is in the glass tube prior to filling the burette. If there is, a preliminary filling and draining of the burette will restore the proper balance of operating pressures.

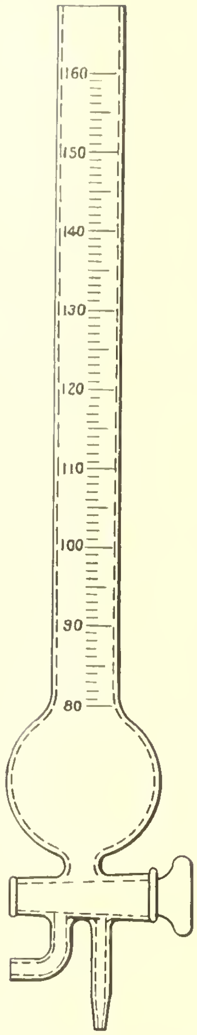


Figure 4.—Suggested burette for automatic measurement of water

MODIFICATION FOR AUTOMATIC OPERATION

The apparatus described above is of most value in laboratories where a large number of batches of mortar are mixed each day. For laboratories doing a large amount of routine testing in which a large number of batches of mortar are prepared each day requiring the same amount of water in each batch, a modification of this apparatus to permit the entirely automatic measurement of water may be used. In this case the same water supply bottle and glass tube are used, but the burette has inverted readings. An example of such a burette is shown in Figure 4. The zero mark is at the glass stop cock, and the 160 cubic centimeter reading at the top of the burette. The enlarged portion of the burette is immediately above the stop cock. In operation, the adjustment should be made so that the desired amount of water will enter the burette. The operator then needs only to drain the burette and turn the stop cock to the filling position, and the same amount of water will again enter the burette.

The same principle may also be applied to apparatus used in measuring water for concrete mixes.

(Continued from page 181)

ratings for the galvanized pipe, to the relatively large influence of the opinion as to whether spelter condition should be rated 90 or 75, and, in Table 15, to the fact that the trend is based on eight ratings rather than on four or five as in preceding tables.

However, Table 14 shows that seven observers are not in disagreement with the trend shown by the average ratings although the ratings of the individuals, being in a number of instances the same for more than one pipe, do not permit of the fine distinctions in order of excellence which result from a grouping on the basis of average ratings. Eight observers are in agreement that the first three pipe in this table are not inferior to either of the last two and all observers agree that the last pipe in the table is not superior to any of the first four.

Table 15 shows that seven observers agree that the first pipe, No. 65, is superior to all others and two additional observers agree that this pipe is at least the equal of any others in the table. Eight observers are in agreement that the first two pipe, if not actually superior, at least are not inferior to any of the others and eight observers agree that the last pipe is not superior to any of the other seven. This agreement between indicated trends of excellence is considered good in view of the small range in average ratings.

CONCLUSIONS AND INDICATIONS OUTLINED

The only definite conclusions which may be drawn as a result of this investigation are:

1. That the rating schedule used in this survey for measuring the relative behavior in service of corrugated sheet-metal culverts is sound in principle.
2. That the use of such a schedule may be expected to yield results having a degree of accuracy sufficient for the purpose and its use by different observers may be expected to yield comparable results.

Having in mind the manifest inadequacy of the data as a basis for any definite conclusions, certain indications applying only to metal culverts installed under the conditions peculiar to this locality appear to be worthy of note. These indications are that from the standpoint of resistance to corrosion:

1. Galvanized pipe is superior to ungalvanized pipe.
2. The group of metals which includes copper-bearing steel, copper-bearing iron and ingot iron is superior to the group which includes open-hearth steel and Bessemer steel.
3. For galvanized pipe the self-cleaning condition appears to be more favorable than the filling condition, while the reverse is true for the black (ungalvanized) pipe. This may have some significance as indicating a difference in the rates of deterioration of spelter coating and base metal under different conditions of service.

RESULTS OBTAINED BY THE USE OF CEMENT

REPORT OF AMERICAN ENGINEERS TO SIXTH INTERNATIONAL ROAD CONGRESS

By FRANK T. SHEETS, *Chief Highway Engineer, Department of Public Works and Buildings of Illinois;* ROY W. CRUM, *Director, Highway Research Board, National Academy of Sciences;* E. M. FLEMING, *Manager, Highways and Municipal Bureau, Portland Cement Association;* ARTHUR N. JOHNSON, *Dean, College of Engineering, University of Maryland;* CLIFFORD OLDER, *Consulting Engineer, Chicago, Ill.*, and RAYMOND E. TOMS, *Principal Highway Engineer, United States Bureau of Public Roads*

At the end of 1928 there had been constructed in the United States on strictly rural highways approximately 59,000 miles of Portland cement concrete pavement, or more than four times the mileage which existed at the end of 1920. During the year 1928 alone, 8,756 miles were built. Such intensive construction has necessarily called for the rapid development of design and construction practices. This report outlines the present methods employed in the design, construction, and maintenance of concrete pavements.

Considerable advance has been made in the structural design of pavements. The report calls attention to certain fundamentals which should be observed in the construction of the subgrade, and reference is made to new developments in the design of the cross section, as well as important features in the proportioning of the reinforcing steel and the construction of expansion joints.

The trend is toward intensive research as to the effect of various characteristics of constituent materials on the quality of the concrete. Recent investigations indicate that the characteristics of the aggregates have more influence on the tensile and flexural strength of the concrete than upon the compressive strength.

The design of concrete mixtures by the water-cement ratio and mortar-void methods is rapidly gaining in favor. To secure the benefits of proper design of mixtures, the ingredients must be accurately measured and the water content closely controlled. These important features are described and references given to more complete information.

Rapid strides have been made in the development of equipment for handling materials and for mixing and placing concrete. Rigid inspection and modern equipment have aided in securing improved quality, smoother surfaces, and better production. The curing problem still needs much study.

The rapidly increasing mileage of concrete pavement has stressed the problem of maintenance. Much study has been given organization, methods, and equipment. Some of the more important methods are described. One new development is the use of high early strength concrete for repair work.

INTRODUCTION

The extensive construction of cement concrete roads in the United States during the past decade has done much to standardize design and construction methods. Great progress has been made in the development of equipment. Complete control of every phase of design and construction in order that pavements of the highest quality may be obtained is now generally recognized.

These reports are designed to summarize the present practice in design, construction, and maintenance methods and have been prepared by engineers whose experience and achievements have gained merited recognition.

STRUCTURAL DESIGN OF CONCRETE PAVEMENTS

By CLIFFORD OLDER

The subgrade.—It is generally recognized that the most vital single consideration in the preparation of the subgrade is uniformity. If necessary to secure this condition old crusts are completely loosened and the material distributed uniformly over the area to be paved or otherwise disposed of. When areas of yielding soil are encountered, it has been found desirable to remove the soft material and replace with earth of the general character of that of the surrounding subgrade.

Tile drains are used to eliminate free water from areas supplied by underground sources. In heavy clay soils a well drained layer of granular material is used by some engineers although its need is not universally recognized.

Rollers weighing from 3 to 5 tons are generally preferred for final finishing and it is usual to prohibit the filling of low spots after final rolling. Excessive rolling and uneven compaction are avoided in so far as feasible.

Inasmuch as the accuracy with which the subgrade surface is finished to the specified contour determines the uniformity of the pavement thickness and strength, much attention is given to this item. It is usual to prohibit high but not low areas in order that the thickness of the pavement may at no point be less than that specified.

Design of cross section.—It is all but universally recognized that the thickness of the mid portion of a concrete pavement need be but about seven-tenths of that near the edges in order that the flexural strength throughout may be approximately balanced. The thickness determined for the edge is usually continued for a distance of about 2 feet and then diminished to 0.7 at a distance from the edge somewhat greater than one-half the standard spacing of wheels on an axle.

It is generally believed that if the edge thickness be proportioned in accordance with the formula $t=\sqrt{\frac{3W}{.5m}}$, reasonable freedom from structural failure will result. In this formula t = edge thickness, W = maximum expected wheel load, and m = the modulus of rupture of the concrete.

Much attention is given to methods of proportioning the ingredients in order to secure the highest possible modulus of rupture consistent with economy of construction. It is customary to prepare sample test beams as the work progresses in order to provide means for checking the final result.

It is obvious that a greater factor of safety would result in using a smaller percentage of "m" in the above formula. It is the author's practice to make an additional reduction of one pound for each foot of slab length or width between edges, in order to allow for the tension set up by subgrade friction during periods of falling temperature.

More nearly rational formulæ for edge, corner, and mid portion thickness are available for design, for which see "Computation of Stresses in Concrete Roads," by H. M. Westergaard, Proceedings of the Fifth Annual Meeting of the Highway Research Board, Part I.

Steel reinforcement and joints.—A comprehensive survey of existing plain and reinforced concrete pavements has shown the advantages that accompany the judicious use of reinforcing steel, for which see Report of Investigation of the Economic Value of Reinforcement in Concrete Roads, by C. A. Hogentogler, Proceedings of the Fifth Annual Meeting of the Highway Research Board, Part II.

The most common method of proportioning reinforcing steel is to assume that a crack may occur halfway between the side or end margins, and that the area of steel throughout should be ample to prevent the crack from widening when the temperature falls and the pavement contracts. Thus the stress in the steel is assumed to equal the weight of one-half the slab times the coefficient of friction of the pavement on the subgrade. This coefficient is usually taken as 1.5 or 2.

It is generally recognized that the principal function of reinforcing steel so proportioned is to cause a dowelling or shear action between the pavement edges across which it passes. Excepting at predetermined joints where stiff dowel bars or other means are often used, this is accomplished more by the roughness of the concrete held in close contact than because of the stiffness of the small reinforcing members usually selected.

It has been determined by experience that in a climate where the ground freezes more than a few inches in depth, irregular longitudinal cracks will appear unless the pavement is divided into strips having a width of about 10 feet or less. In pavements of ordinary width the adjacent strips are held together with deformed dowel bars and the joint is of the tongue-and-groove type.

Transverse expansion joints are inserted in order to avoid excessive compressive stresses when the pavement expands.

It is usual to provide for a movement of about 1 inch per 100 feet of pavement length after taking into consideration the degree of compressibility of the material used in the joint.

It is not generally considered the best practice to concentrate the expansion provisions at wide intervals as in such case subgrade friction may cause troublesome compressive stresses halfway between expansion joints.

Attempts have been made entirely to prevent transverse cracks by forming transverse joints at various intervals.

The Bates road tests and general experience indicate that for heavy traffic roads, at least, the interval between joints must be relatively short if transverse cracks are to be avoided in pavements of ordinary thickness.

MATERIALS

By A. N. JOHNSON

During the past three years, since the last report was presented, the advance that has been made in the art of concrete road construction as the result of extensive researches has increased the use in the field of the water-cement-ratio law, the proportioning by weight of aggregates, and the use of flexural tests as a measure of concrete strength.

The effect of various characteristics of the aggregates upon the concrete is still a matter of investigation.

The injurious effect of organic impurities in fine aggregate is well established, and to detect such impurities 25 State highway departments now prescribe the sodium hydroxide test, which is readily applied in the field. Neither the fine nor the coarse aggregates should contain material that increases greatly in volume under exposure to moisture.

In general, it has been found that while a wide variety of aggregates showed little variation in comparative crushing strength, this is not true of flexural and tensile strength.

Investigations on this subject are mostly recent. A symposium on mineral aggregates was held at the

Thirty-second Annual Meeting of the American Society for Testing Materials in June, 1929. A recent investigation of the effect of various characteristics of aggregates upon the strength of concrete is published in "Public Roads," the magazine of the United States Bureau of Public Roads, June, 1929, as reported by Mr. W. F. Kellermann. These investigations indicated that while the compressive strength may not be greatly affected by the variation in the nature of the aggregates, this is not always the case with the flexural and tensile strength of concrete.

In a report by Mr. R. W. Crum on "The Control of Methods and Mixtures for Concrete for Pavements" presented at the Ninth Annual Meeting of the Highway Research Board, December, 1929, it is stated that for mixtures that contained the same absolute volume of aggregates and cement the strengths were the same. It is further stated that these data do not disclose any constant relation between the transverse and compressive strengths.

Reports from other investigations bear out the conclusion that the characteristics of the aggregates have more influence on the tensile and flexural strength than upon the compressive strength. A recent meeting of the executive committee of the Highway Research Board called attention to this subject:

"The need of coordinated research in this field was strongly emphasized in the Symposium on Aggregates at the recent meeting of the American Society for Testing Materials. It is plain that owing to the wide variations in the natural materials, as well as in the conditions of use in road work, a mass of sound information as to the effects of different characteristics upon results in use is greatly needed. Standard specifications for aggregates can not be much more than forms indicating the important points to be covered. It is necessarily left to the individual user to specify test limits to meet his local conditions as to materials and use. The information needed by him concerning the effects of different characteristics of the aggregates is far from complete at the present time."

It is evident that before we can interpret the results of investigations of the effect of characteristics of aggregates, there must be accumulated more comparative data to determine the practice that will make for the greatest practical economy in road construction.

The use of reinforcement is increasing. Structural steel is generally prescribed (standard specifications of the American Society for Testing Materials, serial designation A 15–14). In addition, some State specifications permit rerolled steel (American Society for Testing Materials specifications, serial designation A 16–14), while mesh reinforcement shall meet the requirements of the American Society for Testing Materials specifications for cold-drawn steel wire (serial designation A 62–27).

Attention is invited to the report of investigation of the effect of steel reinforcement in concrete roads which is to be found in the Proceedings of the Fifth Annual Meeting of the Highway Research Board of the National Research Council, Washington, D. C., December, 1925. This report was made by Mr. C. A. Hogentogler.

The importance of rigid control of the water, cement, and mineral aggregates, both as to quality and quantity, has made closer and more expert inspection necessary. The call is for men as inspectors who have special skill and experience.

DESIGN OF CONCRETE MIXTURES AND METHODS OF PROPORTIONING

By R. W. CRUM

Design.—Specifications for concrete mixtures stated arbitrarily by loose volumes of materials are still in common use in the United States. They are in many cases limited by requirements based upon recognized principles of design, and in some States specifications are based throughout on these principles. The use of specifically designed mixtures in work involving the possibility of using materials from many different sources necessitates flexibility in specifications. The difficulty of incorporating this flexibility into contracts let on competitive bidding delays more general adoption of scientific principles of mixture design on pavement work.

Principles of design are based upon the facts that the strength of concrete depends primarily upon the amount of solid material present in a given vloume of concrete and upon the extent to which the solid particles are bonded together by the cement (*1*).[1] It has been demonstrated that the resultant of these factors is measured by the ratio of absolute volumes of cement to cement plus aggregates or of cement to void space (*2*). Since void space is practically controlled by the amount of mixing water used in plastic mixtures, the ratio of water to cement is also a measure of the strength of any combination of the same materials.

The water-cement-ratio method of design consists in selecting the water-cement ratio corresponding to the required strength and finding the most suitable combination of aggregates to give the desired workability (*3, 4*).

The mortar-void method is based upon the principle that the strength depends upon the ratio of cement to voids in the concrete and that the greatest strength will result for a given void-cement ratio when the amount of mixing water is such that the mortar part of the concrete will occupy the least volume (*5*).

Mixtures of definite amounts of water with given cement, fine and coarse aggregate can be designed by either method to produce concrete of the required strength (*2*).

Each prospective material should be separately studied through laboratory tests in designing mixtures for specific purposes, because a design made for certain materials will not necessarily apply to other materials. The strength of the concrete, particularly flexural strength, may be affected by the quality of the cement used and by such characteristics of the aggregates as shape and size of particle, mineralogical composition, texture, porosity, and strength (*6, 7*).

Workability is a primary factor in design. However, laboratory tests indicative of workability under modern placing and finishing methods on pavement work have not yet been devised. Until some test for field workability is developed the mixtures designed by these methods will be subject to cut-and-try modification in the field.

Proportioning (*measuring*).—In order to secure the benefits of proper design of mixtures it is necessary for the various ingredients to be accurately measured. Measurement of aggregates by weight is rapidly increasing on highway work. It has been found that these materials can be weighed accurately and rapidly without extra cost and that more uniform concrete is produced.

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[1] Figures in parentheses refer to the references given at the end of this section.

The water-measuring tanks on the latest paving mixers give accurate results when supplied with water under constant pressure and free from entrained air. Inaccuracies in water measurement are often caused by frequent changes in pipe-line pressure and the inclusion of air in the water incident to conditions of the work. This trouble has been eliminated by means of a double-tank arrangement for keeping the measuring tank under atmospheric pressure only (*8*).

In order to secure the benefits from accurate measurement of water and aggregates it is necessary that correction be made for the water carried by the aggregates or that which will be absorbed by them. Rapid methods have been devised for making frequent determinations of this moisture factor (*9*).

It is necessary also that attention be given to the uniformity of the aggregates and that weight corrections be made for oversize particles in the fine aggregate and for undersize particles in the coarse aggregate (*9*).

It has been demonstrated that very uniform concrete can be produced in pavement work when each of the details of measurement is faithfully carried out (*9*).

Cement continues to be measured as packed by the manufacturers except when shipped in bulk. One possibility for improving in handling cement lies in protection from incidental losses during loading and transportation.

REFERENCES

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2. Design of Concrete Mixtures, International Congress for Testing Materials, 1927—Crum.
3. Bulletin 1, Structural Materials Research Laboratory, Portland Cement Association—Abrams.
4. Report of Director of Research, Portland Cement Association, 1928—McMillan and Johnson.
5. Bulletin 137, Illinois Engineering Experiment Station—Talbot and Richart.
6. Influence of Quality of Coarse Aggregate on Strength of Concrete, Symposium on Mineral Aggregates, American Society for Testing Materials, 1929—Lang.
7. Effect of Type and Gradation of Coarse Aggregate Upon the Flexural and Tensile Strength of Plain Concrete. Public Roads, June, 1929—Kellerman.
8. Iowa Highway Commission.
9. Control of Materials and Mixtures for Concrete Pavements, Highway Research Board, 1929—Crum.

CONSTRUCTION OF CONCRETE PAVEMENTS

By E. M. FLEMING

Equipment and methods of construction.—Concrete road construction is particularly adapted to quantity production as most of the operations lend themselves readily to the use of machinery. Competition among contractors is so keen that only the best managed and equipped outfits can hope to be uniformly successful.

Locomotive or caterpillar cranes unload the aggregates from freight cars, deposit them in stock piles and at the same time keep adjacent overhead bins filled. From these bins the materials flow by gravity to volume measuring hoppers or to weighing hoppers, suspended below the bins. Trucks or narrow-gage railroad cars are filled by gravity from the hoppers. Weighing aggregates is a comparatively new development and one which promises to supersede other methods of proportioning. Weighing equipment has been refined so that the operation can be performed as quickly as the materials were formerly measured by volume.

In about 85 per cent of the cases, raw materials are transported to the mixer in short wheel-base trucks hauling either one or two batches, including cement. These trucks are equipped with pneumatic tires, should be of rugged construction and easily and quickly dumped. Narrow-gage industrial railroads are also used.

Self-propelling concrete mixers mounted on caterpillar treads and operated by gasoline engines, and turning out approximately 1 cubic yard of mixed concrete every 75 seconds, are the most common. A recent improvement in mixers is the development of a water-measuring device which will control the quantity of water to within 1 pint.

Concrete should be mixed to such a consistency that it will slowly settle when deposited on the subgrade. It is then leveled off a little high with shovels, struck off and compacted by a power finishing machine, after which the surface is floated or scraped with long-handled floats or straightedges, tested for surface smoothness with straightedges and then finally belted. Finishing machines that shape and compact the surface by screeding have practically superseded the older tamping machine. The frequency of passage of the finishing machine over a given area is now reduced to a minimum.

Field control of quality, quantity, and smoothness.—Proper control of these items depends on rigid and intelligent inspection backed by penalties enforceable on the contractor.

Quality of concrete depends on the correct design of the mix, uniform quantities of materials per batch, proper mixing time, adequate curing and the amount of mixing water. Beams and cylinders should be cast at the road and tested at stated periods. Cores should be drilled from the finished slab and tested.

Securing the correct depth of slab depends on good form installation, subgrade preparation and on finishing methods which shape the surface closely to the theoretical section. Forms should be of steel, have adequate bearing and be rigidly pinned in place. The subgrade should be checked for depth by templates immediately before the concrete is placed. Cross sections of the subgrade and of the finished surface taken at the same point at frequent intervals will provide an immediate check. Cores are drilled after the slab has cured, and then measured for depth. Penalties for failure to secure correct depth are assessed against the contractor.

Surface smoothness is a function of correct form construction, concrete of uniform workability and consistency, good finishing methods, and frequent straightedging of the surface. A number of road-building departments now require that surface roughness must not exceed one-eighth inch in 10 feet, although one-fourth inch in 10 feet is still the most common requirement. The contractor is penalized the price of 1 square yard of pavement for each irregularity exceeding the limits.

Curing.—Early curing is accomplished by placing one or two layers of burlap on the green concrete. This is kept damp by light sprinkling.

After 24 hours the burlap is removed and the final curing agent applied. One of the following methods is used:

1. Ponds of water.
2. Wet earth or wet hay cover.
3. Spraying with water.
4. Surface application of calcium chloride.
5. Surface application of sodium silicate.

Water curing is continued for at least eight days. Difficulties in securing proper application of the water have resulted in attempts to find more easily applied curing agents. A recent development is the use of an asphalt or tar seal coat sprayed on the surface immediately after the final belting. This method, as well as the use of calcium chloride and sodium silicate, has met with varying success. Test results and opinions of engineers vary as to the merits of these methods. The use of water is still the one sure method.

MAINTENANCE OF CONCRETE PAVEMENTS

By R. E. Toms

Maintenance of concrete pavements consists of filling cracks or joints, correcting surface depressions, and replacing areas which have disintegrated or been shattered by traffic.

Cracks or joints should be filled with bituminous material once a year or oftener, depending upon the amount of traffic. They should be well cleaned with brooms or compressed air, to a depth of 3 inches, of all dirt, dust, or foreign material before filling. After cleaning they should be filled with hot tar or asphalt by means of pouring cans and covered with clean sand or stone chips to prevent the bituminous material from being picked up by traffic. Wide cracks or joints should be filled with a bituminous mixture, preferably a standard hot-mix bituminous concrete.

Shallow surface depressions may be corrected by painting the depressed area with crack-filling bituminous material and covering with crushed aggregate, ranging from three-quarters inch to one-half inch in size. Successive applications of bituminous material and covering should be used to secure the desired thickness. Surface depressions of greater depth than three-quarters inch should be corrected with bituminous concrete mixtures containing graded coarse aggregate. Such mixtures may be prepared without heating by using an emulsified asphalt or a cut-back tar. Standard hot-mix bituminous concrete is also used for this purpose. The area to be covered by the mixture is usually given a paint coat of tar or asphalt. The bituminous mixture should be tamped into place and every precaution taken to secure a level surface. After compaction the graded coarse aggregate mixtures are usually given a seal coat of tar or asphalt and covered with fine stone chips or coarse sand.

Replacement of areas must be undertaken where the concrete slab is so broken that it will not support traffic when patched with bituminous materials. All broken and disintegrated concrete should be removed and the old slab cut back to good solid concrete. The top edge of the old concrete should be chiseled to a vertical edge for a depth of at least 1 inch below the surface. The remainder of the old concrete face should be left as rough as possible. The subgrade for the patched area should be excavated so that the new concrete will extend several inches under the old slab and have a depth of from 2 to 4 inches greater than the original slab. In soft, spongy soil, granular material such as cinders, gravel, or broken stone tamped into the subgrade serves to reduce capillary action and give added stability.

The concrete used in replacing pavement areas should be machine mixed. A dry mix should be used to prevent subsidence after placing. The time of mixing for dry mixes should not be less than two minutes. The concrete should be thoroughly tamped in place and finished with a screed, template, and wood float to a regular and uniform surface to fit the crown and edge of the surface of the old pavement. A straightedge should be used longitudinally at either end of the patch to insure a smooth junction with the old concrete. Curing should be by means of wetted burlap placed over the patch as soon as it is finished. The burlap may be replaced with an earth covering at the end of 24 hours, or if it can conveniently be kept wet for three or four days no further curing is necessary. The use of calcium chloride integrally with the concrete mix obviates the necessity for curing.

The concrete used in replacing pavement areas should be a standard pavement mix, or if it is not convenient to keep traffic off of the new patch for the time required for the curing and hardening of ordinary concrete, a quick-hardening" concrete may be used. Quick-hardening concrete made of alumnite cement hardens sufficiently in 24 hours to permit it to be opened to traffic. Alumnite cement is mixed in the same proportions and manipulated in the same manner as Portland cement. It hardens in the presence of water. Particular attention must therefore be given to curing. For several hours after setting all exposed surfaces must be kept damp to prevent incomplete hydration. Quick-hardening concrete which attains sufficient strength to be opened to traffic in three days may be made with ordinary cement by approximately doubling the cement content of a standard mix, using a minimum of water, increasing the time of mix and adding calcium chloride with the mix to accelerate the hardening. These factors together with high atmospheric temperatures for curing enable major repairs to be undertaken with ordinary cement and opened to traffic in comparatively short periods.

Where a large mileage of concrete pavement is to be maintained the maintenance operations are usually performed by two different units; one handling bituminous materials for filling cracks and joints and correcting surface depressions, and the other doing major repairs such as replacing pavement areas. A two or three ton truck and a portable asphalt heater together with small tools constitute the equipment generally used for filling cracks and joints and correcting surface depressions. A supply of bituminous material and covering material may be carried on the truck, or a light truck used to supply these materials from convenient storage places along the road. If standard hot-mix bituminous concrete is used it is usually obtained as needed from asphalt plants operating in near-by cities. For major repairs the organization is less mobile and more equipment is required. A portable mixer of one or two bag capacity, together with the necessary trucks to haul the cement, fine and coarse aggregate from the nearest source of supply or delivery point will be required. A water tank mounted on a truck should be provided also, as water is not always available at the locations of the replacements. The work of an organization of this kind is generally confined to selected locations instead of attempting to cover a designated mileage of pavement as is usually the case with the lighter and more flexible units used for crack filling and correcting surface depressions.

U. S. GOVERNMENT PRINTING OFFICE: 1930

GASOLINE TAXES—FIRST HALF OF 1930

Total tax earned on motor vehicle fuel, etc., refunds, disposition of fund, and gallons taxed

[From reports of State authorities]

State	Gross tax assessed prior to deduction of refunds	Exemption refunds: (Deducted from gross tax)	Total tax earnings on fuel for motor vehicles	Other receipts under tax law (licenses)	Grand total earnings (tax and other receipts)	Collection costs	Disposition of grand total earnings				Tax rates, 1930		Net gallons of gasoline taxed and used by motor vehicles
							Construction and maintenance of rural roads		State and county roads bond payments [1]	For miscellaneous purposes	Cents per gallon		
							State highways	Local roads			Jan. 1	June 30	
Alabama	$3,423,724		$3,423,724	$1	$3,423,725	$17,647	$767,277	$1,699,822	$938,979		4	4	85,593,106
Arizona	1,524,516	$182,442	1,342,074	6	1,342,080	(2)	853,509	488,571			4	4	33,551,860
Arkansas	3,157,552	275,000	2,882,552		2,882,552	67,500	701,786	844,516	[3] 1,268,750		5	5	57,651,044
California	18,040,284	1,797,792	16,242,492		16,242,492	25,749	10,811,162	5,405,581			3	3	541,416,391
Colorado	3,161,987	207,797	2,954,190		2,954,190	[4] 24,955	2,050,464	790,894		[5] $87,877	4	4	73,854,743
Connecticut	2,107,560	[6] 76,645	2,030,915	39,578	2,070,493	(7)	2,070,493				2	2	101,545,770
Delaware	493,691	23,425	470,266		470,266	(8)	470,266				3	3	15,675,528
Florida	7,229,997		7,229,997	10,420	7,240,417	7,051	2,409,999	401,667	[9] 2,409,999	[10] 2,011,701	6	6	120,499,948
Georgia	6,432,781		6,432,781		6,432,781	4,200	4,285,721	1,071,430		[11] 1,071,430	6	6	107,213,018
Idaho	1,250,557	76,741	1,173,816	477	1,174,293	6,217	1,168,076				4	5	[12] 24,486,855
Illinois	13,377,556	444,623	12,932,933		12,932,933	[13] 267,551	8,443,588	4,221,794			3	3	431,097,782
Indiana	8,524,791	533,749	7,991,042		7,991,042	19,157	5,978,913	1,494,729		[14] 498,243	4	4	199,776,038
Iowa	5,458,225	462,328	4,995,897		4,995,897	16,355	2,941,542	2,038,000			3	3	166,529,913
Kansas	5,086,834	592,513	4,494,321		4,494,321	(15)	3,644,321	850,000			3	3	149,810,701
Kentucky	3,870,979		3,870,979	2,889	3,873,868	12,472	3,861,396				5	5	77,419,583
Louisiana	3,594,441	479	3,593,962		3,593,962	(16)	3,033,962		560,000		4	4	89,849,038
Maine	1,602,424	213,483	1,388,941	[17] 53,371	1,442,312	32,352	881,225	528,735			4	4	34,723,518
Maryland	3,306,579	117,774	3,188,805		3,188,805	4,500	2,517,444			[18] 666,861	4	4	79,720,113
Massachusetts	4,820,680	66,343	4,754,337		4,754,337	(19)	[20] 4,754,337				2	2	237,716,821
Michigan	11,046,931	1,077,343	9,969,588	395	9,969,983	23,889	5,800,584	2,927,292	1,182,654	[21] 35,564	3	3	332,319,601
Minnesota	5,052,173	378,434	4,673,739		4,673,739	(22)	3,115,826	1,557,913			3	3	155,791,314
Mississippi	3,373,150		3,373,150	[23] 62,273	3,435,423	2,700	1,314,705	2,020,270		[23] 97,748	5	5	67,463,005
Missouri	4,122,351	102,613	4,019,738		4,019,738	26,195	3,993,543				2	2	200,986,879
Montana	1,618,701	257,810	1,360,891		1,360,891	12,071	1,348,820				5	5	27,217,819
Nebraska	4,282,605	34,080	4,248,525		4,248,525	3,750	3,183,581	1,061,194			4	4	106,213,125
Nevada	329,932	30,915	299,017		299,017		299,017				4	4	7,475,422
New Hampshire	1,029,187	47,204	981,983		981,983		736,487		245,496		4	4	24,549,580
New Jersey	4,967,626		4,967,626	13,608	4,981,234	8,000	4,928,234			[24] 45,000	2	2	248,381,311
New Mexico	1,277,033		1,277,033	4,557	1,281,590	25,632	823,958		432,000		5	5	25,540,650
New York	13,396,516	275,899	13,120,617		13,120,617	[25] 133,965	9,702,489	2,587,330		[26] 696,833	2	2	656,030,864
North Carolina	6,281,527	322,662	5,958,865		5,958,865	6,000	2,111,692	1,191,173	2,650,000		5	5	119,177,300
North Dakota	1,237,078	453,978	783,100		783,100	18,100	510,000	255,000			3	3	26,103,326
Ohio	18,652,809	900,495	17,752,314	315	17,752,629		11,095,393	3,550,526		[27] 3,106,710	4	4	443,807,845
Oklahoma	6,187,523	11,558	6,175,965		6,175,965	(28)	4,631,974	1,543,991			4	4	154,399,130
Oregon	3,047,008	255,598	2,791,410		2,791,410	6,015	2,785,395				[29] 4	[29] 4	[30] 70,042,779
Pennsylvania	16,045,671	93,084	15,952,587	[31] 187,478	16,140,065	[32] 170,024	12,475,968	1,994,073	1,500,000		4	[33] 4	398,814,666
Rhode Island	814,393	18,326	796,067	372	796,439	(34)	601,911		194,528		2	2	39,803,346
South Carolina	3,389,057	12,809	3,376,248	751	3,376,999	(35)	2,814,166	562,833			6	6	56,270,799
South Dakota	2,052,487	787,747	1,264,740		1,264,740	5,250	871,501		387,989		4	4	31,618,494
Tennessee	4,941,567		4,941,567		4,941,567	24,708	2,950,115	983,372	983,372		5	5	98,831,[illegible]
Texas	15,383,741	1,320,670	14,063,071		14,063,071		10,547,303			[36] 3,515,768	4	4	351,576,[illegible]
Utah	1,000,078		1,000,078	476	1,000,554	1,727	611,877		386,950		3½	3½	28,573,651
Vermont	716,720		716,720		716,720	(37)	716,720				4	4	17,918,00[illegible]
Virginia	5,203,712	301,971	4,901,741		4,901,741	16,987	3,256,503	1,628,251			5	5	98,034,819
Washington	3,597,870	319,023	3,278,847		3,278,847	(38)	2,185,898	1,092,949			3	3	109,294,898
West Virginia	2,442,012	97,600	2,344,412	4,677	2,349,089		621,652		1,727,437		4	4	58,610,300
Wisconsin	3,835,680	323,594	3,512,086		3,512,086	5,000	1,291,419	1,962,610		[14] 253,057	2	2	175,604,292
Wyoming	558,211		558,211		558,211		418,558	139,653			4	4	13,955,258
District of Columbia	751,495	5,000	746,495		746,495					[14] 746,495	2	2	37,324,724
Total			230,600,455	381,644	230,982,099	995,719	157,390,770	44,894,169	14,868,154	12,833,287	Average rate 3.39		6,809,863,076

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[1] For state highway bonds, except as noted.

[2] From State highway fund, approximately $7,000.

[3] Divided between State and county road bonds, approximately 50 per cent to each.

[4] Includes all expenses of State inspector of oils.

[5] For town and city streets.

[6] Exemption made at time of sale.

[7] State appropriation of $14,000 deducted from motor vehicle fees.

[8] Appropriation for State treasurer's office.

[9] For county and town bonds.

[10] Includes $2,008,332 for public schools: remainder, $3,369 reserve for gasoline tax collection.

[11] For public schools.

[12] Includes 5,052,717 gallons at 4 cent tax, and 19,434,138 gllaons at 5-cent tax.

[13] 2 per cent of gross tax receipts allowed to distributors.

[14] For city streets.

[15] From State general fund, $7,500 for half year.

[16] State appropriation, $3,750 for half year.

[17] Tax of 1 cent on 5,337,079 gallons of gasoline sold for uses other than for motor vehicles.

[18] Includes $629,361 for Baltimore streets, and $37,500 for oyster conservation work.

[19] Reported as "no cost."

[20] Credited to highway fund, which is later appropriated to State highways, local roads and State bond payments.

[21] Includes $35,169 for city streets, and $395 to State general fund from dealers' licenses.

[22] From State treasury, as part of inspection service Appropriation about $8,250.

[23] For sea-wall protection of road: Partly paid by special gasoline tax of adjacent counties, amounting to $62,273 (other receipts).

[24] For inland waterways under State Department of Commerce and Navigation.

[25] 1 per cent of gross tax receipts allowed distributors for collection of tax: In addition $28,126 from State general fund for administration.

[26] For New York City, $646,833, and for refund reserve $50,000.

[27] For streets of townships and municipalities.

[28] From State general fund by appropriation.

[29] "Distillate" taxed 3½ cents per gallon.

[30] Includes about 1,949,450 gallons of distillate.

[31] Consists of delinquent taxes of previous years, with fines and penalties.

[32] Dealers allowed commission of 1 per cent on first $1,000 of tax collected, and one-half per cent on remainder.

[33] Gasoline tax rate reduced to 3 cents on July 1.

[34] From motor vehicle fund, $5,000.

[35] State tax commission appropriation.

[36] For free school fund.

[37] From motor vehicle fund, $1,000.

[38] From motor vehicle fund, $2,500.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

OCTOBER 31, 1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage	Total			Initial	Stage[1]	Total		
Alabama	2,196.6	$ 1,457,591.49	$ 714,054.55	70.9	8.2	79.1	$ 1,981,827.41	$ 926,375.44	82.7	8.8	91.5	$ 6,722,926.91	Alabama
Arizona	983.0	3,492,782.60	2,640,927.84	107.6	131.1	238.7	853,163.90	811,619.79	48.3		48.3	2,460,561.70	Arizona
Arkansas	1,773.2	6,370,755.11	2,981,601.94	204.6	45.1	249.7	1,634,640.99	806,688.06	40.7	18.7	69.4	2,752,806.02	Arkansas
California	1,908.0	9,307,726.11	4,063,411.74	227.4	19.0	246.4	786,816.53	285,411.29	1.7	13.1	14.8	4,619,406.63	California
Colorado	1,236.6	4,556,622.93	2,439,302.71	156.0	70.4	226.4	986,791.05	543,673.67	27.8	29.7	67.5	3,951,911.03	Colorado
Connecticut	243.3	3,084,889.11	1,294,964.40	19.4		19.4	668,308.20	131,085.00	8.7		8.7	1,262,846.45	Connecticut
Delaware	273.0	884,880.75	427,483.87	45.9		45.9	167,677.86	83,838.97	8.0		8.0	617,586.72	Delaware
Florida	633.8	4,680,361.00	2,229,234.31	76.0	6.5	81.6	438,506.31	204,991.28	21.7		21.7	2,689,853.34	Florida
Georgia	2,639.8	6,279,884.98	2,994,627.24	169.8	123.8	283.6	1,211,743.13	524,234.81	29.1	24.4	63.6	5,265,235.14	Georgia
Idaho	1,248.8	2,785,414.78	1,675,724.22	182.5	14.8	197.3	417,697.04	248,500.00	29.3	8.2	37.5	1,719,413.88	Idaho
Illinois	2,101.9	21,438,919.36	9,686,063.83	613.1	59.8	672.9	3,018,350.10	1,386,847.33	85.2	12.6	97.9	7,032,824.13	Illinois
Indiana	1,500.9	4,560,726.70	2,213,573.73	143.4		143.4	903,188.23	375,647.33	26.2		26.2	6,311,033.11	Indiana
Iowa	3,113.2	3,553,462.98	1,481,309.20	28.6	92.7	121,3						3,342,943.65	Iowa
Kansas	2,893.8	6,317,860.90	2,488,703.63	254.4	53.3	307.7	808,606.93	393,031.84	29.5	36.7	66.2	4,383,034.41	Kansas
Kentucky	1,362.6	8,470,356.91	3,694,645.55	177.8	190.3	368.1	1,241,963.99	482,852.17	13.0	59.6	72.6	1,937,610.38	Kentucky
Louisiana	1,397.3	4,177,364.05	2,018,690.73	150.8	20.9	171.7	872,117.68	396,929.89	31.1	7.1	38.2	2,698,458.35	Louisiana
Maine	655.9	2,861,316.12	1,065,232.79	71.0		71.0	974,630.81	399,268.09	8.4		8.4	1,899,835.21	Maine
Maryland	647.4	1,960,432.02	935,710.32	37.8	16.9	64.7						1,036,836.05	Maryland
Massachusetts	698.2	5,636,825.45	1,329,640.32	54.6		54.6	806,166.36	358,639.90	3.8		3.8	3,167,644.98	Massachusetts
Michigan	1,683.9	10,544,301.27	4,206,612.83	240.8	28.9	269.7	1,760,614.80	779,175.00	53.6	6.4	60.0	4,719,842.41	Michigan
Minnesota	4,038.0	9,493,816.06	3,373,053.71	149.1	232.8	381.9	500,431.65	213,000.00	10.8	42.5	53.3	3,299,753.06	Minnesota
Mississippi	1,839.8	639,084.13	319,245.57	36.1	7.7	43.8	319,042.54	159,521.27		13.6	13.6	5,592,293.28	Mississippi
Missouri	2,541.1	8,286,692.51	2,785,739.32	131.7	53.7	185.4	3,650,885.87	1,363,983.40	64.7	26.0	90.7	3,451,438.80	Missouri
Montana	1,816.2	7,528,253.63	4,243,719.44	587.2	40.9	628.1	1,467,029.39	807,318.61	125.4	19.3	144.7	4,858,502.96	Montana
Nebraska	3,781.1	5,038,913.07	2,402,338.70	254.1	128.8	382.9	1,123,809.33	471,800.80	28.6	29.3	57.9	4,210,666.51	Nebraska
Nevada	1,229.2	1,064,373.70	923,821.23	23.2	112.1	135.3	322,783.14	286,792.78	29.2	.7	29.9	1,887,644.63	Nevada
New Hampshire	364.9	1,699,321.63	685,708.93	35.0	1.6	36.6						598,242.63	New Hampshire
New Jersey	633.4	4,379,066.90	1,093,726.32	41.2		41.2						2,547,196.76	New Jersey
New Mexico	1,938.0	3,772,899.14	2,542,770.74	195.4	66.3	251.7	699,518.26	493,498.20	14.8	1.7	16.5	2,428,310.58	New Mexico
New York	2,589.9	23,384,721.76	4,510,305.00	301.3		301.3	4,167,316.00	987,595.60	71.4		71.4	12,197,655.54	New York
North Carolina	1,878.9	3,571,160.99	1,732,884.07	136.6	28.8	165.4	645,865.88	317,731.43	43.0	9.2	52.2	4,469,803.59	North Carolina
North Dakota	4,400.3	2,424,190.93	1,247,330.77	327.7	201.2	528.9	977,253.47	488,332.92	130.8	104.6	235.4	2,647,096.11	North Dakota
Ohio	2,317.9	21,111,281.88	7,049,364.05	356.6	35.9	392.4	1,080,353.50	324,569.00	9.6		9.6	4,647,759.25	Ohio
Oklahoma	1,872.6	6,290,897.80	2,977,428.53	191.8	97.4	289.2	114,374.41	69,565.37	15.6		15.6	2,899,336.18	Oklahoma
Oregon	1,165.2	6,208,963.98	3,657,807.07	262.9	85.2	348.1	300,260.40	182,772.27	19.5		19.5	1,941,114.05	Oregon
Pennsylvania	2,567.8	12,537,921.09	3,800,548.95	118.4		118.4	1,398,175.36	546,887.13	18.3		18.3	6,359,238.34	Pennsylvania
Rhode Island	200.2	2,251,318.84	917,014.74	35.8		35.9						619,426.42	Rhode Island
South Carolina	1,842.2	6,107,146.93	2,347,085.69	115.3	116.5	231.9	1,169,195.41	657,871.88	9.4	26.1	35.5	1,293,875.39	South Carolina
South Dakota	3,631.3	4,039,291.15	2,193,748.37	351.0	96.3	447.3	843,395.18	489,101.48	83.5	65.3	138.8	2,527,887.38	South Dakota
Tennessee	1,302.2	4,490,569.10	2,016,498.08	180.1	43.9	224.0	986,185.36	491,112.39	16.8		16.9	3,350,733.27	Tennessee
Texas	6,836.6	14,766,525.06	6,147,590.87	515.2	214.4	729.6	2,820,495.95	1,105,639.59	110.7	46.8	157.6	10,488,476.78	Texas
Utah	973.0	1,587,554.23	1,150,181.01	114.9	28.5	143.4	763,297.13	549,291.41	22.9	71.4	94.3	1,492,728.52	Utah
Vermont	274.6	1,572,665.78	483,333.34	29.3	2.6	31.9						624,276.71	Vermont
Virginia	1,546.4	4,600,321.27	2,195,470.61	196.3	6.0	202.3	1,020,020.56	453,299.43	32.6	14.2	46.8	2,267,039.96	Virginia
Washington	978.6	2,798,789.14	1,231,100.00	49.9	36.3	86.2	699,621.18	224,900.00	14.7		14.7	2,868,191.86	Washington
West Virginia	709.3	6,063,098.32	1,846,479.98	120.3	41.3	161.6	851,788.19	363,021.27	9.7		9.7	1,093,807.72	West Virginia
Wisconsin	2,293.6	8,669,758.50	3,496,816.95	212.4	52.8	265.2	1,287,640.85	574,000.00	37.8		37.8	2,624,034.41	Wisconsin
Wyoming	1,680.1	2,791,682.28	1,852,902.71	212.3	120.2	332.6	190,568.49	125,108.19	12.0	15.2	27.2	1,731,460.10	Wyoming
Hawaii	41.2	882,655.31	374,008.63	21.7		21.7	197,730.75	98,865.37	9.0		9.0	1,852,769.59	Hawaii
TOTALS	86,074.1	288,265,306.63	120,070,429.03	8,325.1	2,721.9	11,047.0	47,118,849.46	20,872,289.45	1,489.6	701.1	2,190.7	162,263,168.78	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 10 ▽ DECEMBER, 1930

CONSTRUCTING A MIXED-IN-PLACE SURFACE

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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VOL. 11, NO. 10 **DECEMBER, 1930** **G. P. St. CLAIR, Editor**

TABLE OF CONTENTS

Page

Bituminous Treatments Used on Roads of Intermediate Type in the Western States . . . 189

The Most Recent Methods Adopted for the Use of Tar, Bitumen, and Asphalt in Road Construction. Report of American Engineers to the Sixth International Road Congress . 201

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BITUMINOUS TREATMENTS USED ON ROADS OF INTERMEDIATE TYPE IN THE WESTERN STATES

Reported by **J. T. PAULS**, Senior Highway Engineer, U. S. Bureau of Public Roads

THE progress which western engineers have made and are making in improving their highways with the limited funds available should be of interest to the highway officials of the more densely settled areas. Low-cost bituminous treatments of soil, gravel, and broken-stone roads constitute, in a large measure, the methods adopted by the Western States for improving their important highways to meet the demands of modern traffic. Although the traffic carried by these roads seldom equals that of the main highways of the East, it probably approximates that of the secondary systems, and consequently some of the methods now used satisfactorily in the West may prove suitable in the East for roads of the secondary class.

The material contained in this paper was obtained by the writer during an inspection made in the early summer of 1929 in company with representatives of the Bureau of Public Roads, the asphalt industry, and highway officials of the States visited, which were Nebraska, Wyoming, Colorado, New Mexico, Arizona, California, Nevada, Oregon, Idaho, Washington, and Utah.

The purpose of the inspection was to obtain more complete information on the design, construction, maintenance, and behavior of the different types of bituminous treatments used in the Western States. As there is a considerable difference of opinion regarding various details of these types of improvement, it was considered desirable to visit a large number of projects representing a wide range of conditions and to obtain the views of highway officials familiar with this work.

The predominant types of bituminous treatments used in the West are:

1. Oil treatment of crushed stone and gravel roads by the mixed-in-place and plant-mixed methods.
2. Oil treatment of sandy soil roads by the mixed-in-place method.
3. Treatment of sandy soil roads with light oil by a method involving both penetration and mixing.
4. Surface treatment of bonded stone roads and oil-mixed surfaces.

The light oil, mixed-in-place treatment with crushed gravel and stone aggregate predominates in all the Western States except Oregon, and possibly Washington, where surface treatment is used extensively.

Plant mixing has been used to some extent, but its general adoption is doubtful for construction using light-oil mixes. However, this method is considered to have advantages in many cases over field mixing, particularly where the road to be surfaced has no aggregate in place or where the work is to be done by contract. The premix method in such cases does not cost much, if any, more than the road mix and, in addition, has the advantage of making possible better control of the mixed proportions.[1]

Surface treatments are being used extensively in northern California, Oregon, and Washington and are also gaining in favor in other States where climatic conditions are not particularly favorable to the light-oil-mixed type of surfacing. They are also coming into extensive use in the maintenance of oil-mixed roads.

Some oil mixing on fine sandy soil roads has been done recently in Nebraska and Wyoming. This work, however, has not been extensive and as yet has not passed the experimental stage, but should it prove successful a large field would be opened for its use in these States as well as in some of the others.

Treatment of soil roads with light oil by a method somewhat different from that used in Wyoming and Nebraska has proved successful in a few of the semiarid counties of California. This method is quite similar to that used in Illinois, in that very little mixing is done. More strictly, penetration, together with the kneading action of traffic, is depended upon to combine the oil and the soil.

THE MIXED-IN-PLACE TREATMENT OF CRUSHED GRAVEL AND STONE ROADS

ALTHOUGH the methods and equipment used in the surface mixing process must necessarily be flexible to meet local conditions, the same general principles are used regardless of the materials involved. The aggregate to be oiled is obtained either by scarifying the old base or by adding new material, or by both scarifying and adding new material. The bituminous material is spread in applications of about one-half gallon per square yard by pressure distributors until the desired amount has been spread. Disking immediately after each application is the general practice, and is a very effective means of preliminary mixing, greatly reducing the amount of blading required. If the road is being built without detouring traffic, disking also prevents the freshly applied oil from adhering to passing vehicles.

After the required amount of oil has been applied the materials are mixed with the aid of blade machines by repeated windrowing and spreading until all the aggregate is covered and the mix presents a uniform appearance. Areas which appear lean are given an additional application of oil and are then remixed. An excess of bitumen can be corrected by bringing additional aggregate into the mix. Upon the judgment of the engineer mainly rests the determination of the proper amount of oil. Uniformity of distribution is easily obtained with most of the present-day distributors of experienced operators.

METHODS ARE ADAPTED TO LOCAL CONDITIONS

Mixing may be modified to meet special conditions. As an illustration, one-half the road width may be built at a time, or mixing may be carried out full width and half depth, or the full section may be mixed as one unit.

---

[1] Since the above was written there has been developed a self-propelled mixer, which was used to a limited extent during the construction season of 1930, and which is intended to combine the advantages of field and plant mixing. This machine utilizes an elevator for picking up the aggregate from a windrow on the road and delivering it to a weighing device, from which it passes to the mixer, where it is mixed with the proper amount of road oil. The mixed material is discharged in a windrow behind the machine, where it is spread with a blade in the usual manner. This equipment, if it proves successful, will improve the control of the proportions of the mix and will eliminate other construction difficulties at present attendant on mixed-in-place construction. A photograph of this machine is shown on p. 191.

Mixed-in-Place Surface Constructed of Crushed Stone and 94 Per Cent Road Oil Cut Back with Kerosene

When the mixing process has been completed the mix is spread to uniform cross section for compaction, which may be effected either by rolling or by traffic, or by both. When heavy bituminous materials are used, it is generally advisable to roll the surface; but in the case of light-oil mixes, compaction by traffic seems to give the best results if the surface is lightly bladed or dragged and all irregularities are corrected during the compaction stage.

The use of side forms to obtain a thickened edge is not universal, as the cost is relatively high. On many projects the edges have been thickened either by building up the shoulders or by cutting a V-shaped trench along the sides just prior to spreading the mix.

Obviously the methods just described can not be employed during cold or wet weather, or when the aggregates contain an appreciable amount of moisture. Many projects, no doubt, have been constructed without particular attention being paid to this detail, the importance of which is now generally recognized.

When the existing road metal is of sufficient thickness it is a common practice to scarify the surface to obtain material for the mixing. When this is done it is generally necessary to add aggregate of some particular size to effect a better grading. The combined thickness of the base and the oil-mixed wearing surface varies, but usually is made not less than 6 inches. Adding graded aggregate preparatory to mixing generally results in greater uniformity with a correspondingly better oil-mixed surface than is obtained by scarifying the old stone surface, which, moreover, very often contains clayey material that is not considered desirable for oil-mixing. Loosening the old surface doubtless reduces the supporting value of the road.

Some of the States add clay binder in the construction of the stone surface to insure a better bonded surface. A surface in this condition they find less expensive to maintain than one less firmly bonded, such as is obtained where the fines are granular material rather than clay particles. For this reason clay is often advocated for binder when the metaled road is not to be oil mixed in the near future. When, later, oil mixing is resorted to, the best results are obtained by the addition of new aggregate, the grading of which should be as uniform as possible throughout the project.

CHARACTERISTICS OF BINDER AND AGGREGATE DISCUSSED

The following limits are typical and are considered satisfactory for the binder material in the aggregate to be oil mixed:

Cementing value, not less than 100 (Page cementation test).
Moisture equivalent, not more than 20 per cent.
Lineal shrinkage, not more than 5 per cent.

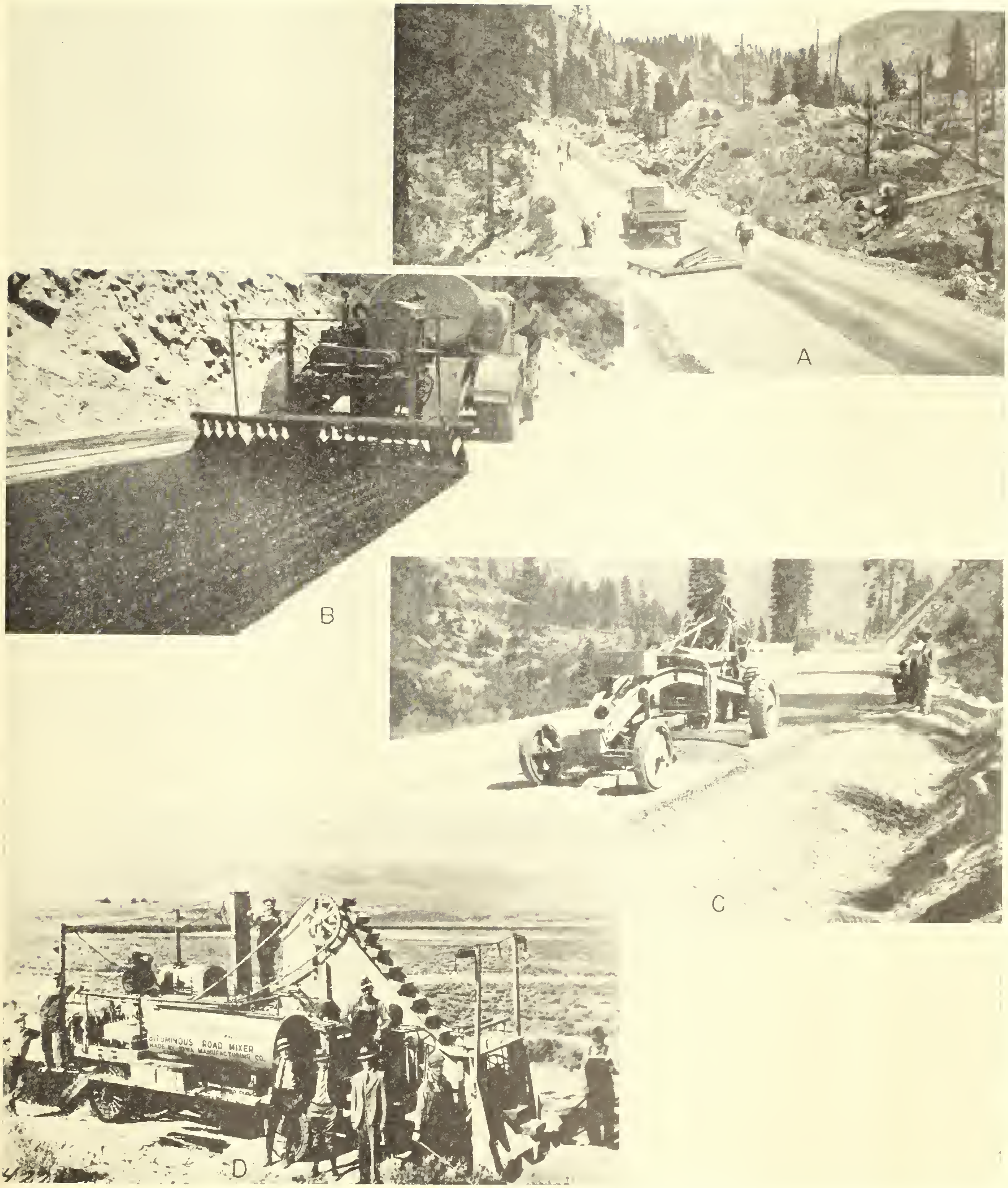
The grading of the aggregate for the oil-mixed surface should be such as to give high stability. With the use of light oil, lacking in body and adhesive properties, this is very important. With oils of this type it is necessary to use aggregate of small size. The maximum size used ranges from about 1 inch to three-fourths inch.

The tendency of the coarser particles of aggregate to become dislodged from the surface and be thrown to the edges was observed on many of the oil-mixed roads. Although this was due in many cases to an insufficient amount of oil, it seemed to be more noticeable on surfaces built with coarser aggregate. A smaller maximum size, of about one-half inch, might have reduced the segregation and raveling noted. However, the primary cause of such failures is believed to be the use of dry mixes.

Table 1 gives the grading of the crushed aggregate used on a large number of typical oil-mixed projects.

SUITABILITY OF AGGREGATES FOR OIL MIXING AN IMPORTANT CONSIDERATION

Some of the Western States, particularly Arizona, have encountered some difficulty in applying the oil-mix treatment in certain areas, due to the fact that the aggregate contains a material known locally as caliche,



Examples of Road-Building Equipment Used on Intermediate Type Roads in the West. A—Brooming the Cover Stone on Surface Treatment Work. B—Attachable Spreader Equipment Used in California. C—Blading the Surface Treatment Cover. D—Portable Mixing Plant Recently Designed and Used on Mixed-in-Place Construction

TABLE 1.—*Grading of crushed aggregate used on typical oil-mixed projects*

State	Average mechanical analysis of road metal on typical oil-mixed projects; per cent by weight passing—							Oil used, per cent by weight	Oil required by formula [1]
	1-inch	½-inch	10-mesh	40-mesh	80-mesh	100-mesh	200-mesh		
Wyoming	95.7	59.9	39.3	23.7	15.2	------	6.7	4.1	3.0
	97.5	75.2	52.5	27.6	15.1	------	6.5	4.3	3.2
	98.0	82.1	63.9	33.5	18.7	------	7.2	4.8	3.5
	92.8	59.1	44.2	34.1	24.3	------	8.9	5.0	3.4
	99.0	78.5	53.9	22.0	11.1	------	5.1	3.9	3.0
	95.4	76.7	63.3	36.0	23.1	------	10.0	4.6	3.9
	91.3	59.0	42.1	26.8	18.0	------	4.9	3.2	2.5
	92.5	65.5	47.6	24.1	15.2	------	7.0	4.9	3.2
Average	95.3	69.5	50.8	28.5	17.6	------	7.0	4.4	3.2
New Mexico	99.8	71.3	59.3	------	------	14.0	7.8	4.0–5.2	3.5
	98.3	79.3	60.9	------	------	16.2	9.0	4.8–6.0	3.7
	98.7	69.8	47.3	------	------	6.1	3.3	3.9	2.7
	99.4	68.5	48.2	------	------	9.5	4.8	3.4	2.9
	98.4	88.8	74.1	------	------	9.3	4.8	---------	3.3
Average	98.9	75.5	58.0	------	------	11.0	5.9	---------	3.2
Idaho	------	48.6	34.0	------	------	------	6.4	4.2	2.9
	------	41.8	28.0	------	------	------	4.8	3.2	2.6
	------	47.7	34.7	------	------	------	5.9	3.2	2.8
	------	42.2	27.9	------	------	------	5.9	3.8	2.7
	------	55.1	35.3	------	------	------	4.6	3.5	2.6
	------	60.1	40.5	------	------	------	7.1	5.0	3.1
	------	40.0	30.1	------	------	------	5.9	3.0	2.8
	------	50.9	32.0	------	------	------	5.7	4.4	2.8
	------	56.4	39.1	------	------	------	4.4	4.9	2.7
	------	54.0	41.8	------	------	------	7.1	4.2	3.1
	------	57.7	48.8	------	------	------	7.5	4.2	3.3
	------	53.0	43.7	------	------	------	6.8	4.1	3.1
	------	56.0	40.4	------	------	------	5.2	4.4	2.8
	------	52.0	42.0	------	------	------	9.3	3.9	3.4
	------	48.2	40.4	------	------	------	5.0	4.3	2.8
	------	46.4	37.4	------	------	------	5.8	3.0	2.9
Average	------	50.6	37.3	------	------	------	6.1	4.0	2.9
Utah	------	------	38.3	------	------	------	10.3	4.4	3.5
	------	------	36.4	------	------	------	8.8	3.8	3.3
	------	------	25.9	------	------	------	6.0	3.3	2.7
	------	------	44.7	------	------	------	7.9	3.6	3.3
	------	------	33.5	------	------	------	5.7	3.2	2.8
	------	------	34.1	------	------	------	5.9	3.6	2.8
Average	------	------	35.5	------	------	------	7.8	3.7	3.1

[1] McKesson and Frickstad formula for estimating the amount of oil required. See text.

which is a calcareous clay or argillaceous limestone. Its presence is often not apparent, especially when it occurs in particles of small size. It has been found impracticable to apply the oil-mix treatment to aggregate containing any appreciable amount of this material.

TESTS DEVISED TO DETERMINE SUITABILITY

J. W. Powers, of the Arizona Highway Department, has devised a test to determine the suitability of an aggregate for oil mixing. His method is to mix the aggregate passing the 10-mesh sieve with sufficient oil to give a predetermined stain. The mixture is then compressed in a 3-inch ring, 2 inches deep, under a load of 2,000 pounds per square inch. The container and sample are then immersed in water and observed. The amount of swelling is indicative of the results which may be expected with the particular aggregate under consideration. Experience has shown that if no swelling of the mix occurs the aggregate will be satisfactory under all conditions and with all oils. A swelling of as little as one-sixteenth inch indicates that only mediocre success may be expected. With aggregate otherwise considered entirely unsuitable for mixing, swelling of as much as three-fourths inch is often obtained.

Preference on the part of the aggregate for the adsorption of water or of asphaltic residues is believed to indicate the probable behavior of an oil-mixed surface subjected to moisture. This principle, it is thought, accounts for the swelling of the material in this test.

Another test for the suitability of the fine aggregate has also been devised by Mr. Powers and is now being used by Mr. Hveem, of the California State Highway Department, in conjunction with research tests on aggregates for oil mixing.

The method of making this test is essentially to combine 10 grams of the fines passing the 200-mesh sieve with 50 cubic centimeters of the 60 to 70 per cent road oil, which has previously been heated to 130° F. Mixing is generally done with an electric mixer of the fountain type, a glass jar of about 250 cubic centimeters capacity being used for the container. The mixing is continued for five minutes, after which 100 cubic centimeters of water at 130° F. is added, followed by five minutes of additional mixing.

In this test unsatisfactory dust will separate from the oil and be deposited at the bottom of the jar, whereas a satisfactory material will remain coated in the oil.

VARIOUS LIGHT-OIL PRODUCTS USED

The petroleum products used in the oil-mix treatments of the West have come from a large number of fields, chiefly in California and Wyoming. So far neither service behavior nor laboratory tests have shown the source of the oil or the method of manufacture to have any important effect on this type of construction.

The various light-oil products coming from the refineries have been obtained by straight topping of the crude petroleum, as residues from steam distillations, and as by-products of the different cracking processes. The oil most widely used contains 60 to 70 per cent asphalt of 80 penetration, and has a specific viscosity (Engler at 122° F.) of 10 to 25, 25 to 50, and 50 to 80 for the light, medium, and heavy grades, respectively. The medium and high viscosity materials are preferred, as the more fluid materials do not produce mixes which are as well bonded as those produced by oils of greater viscosity. The heavier grades are commonly heated, particularly during cool weather, in order to promote ease of application.

SPECIFICATIONS AGREED UPON BY WESTERN STATE OFFICIALS

As a step toward standardizing and simplifying the requirements for road oils, a conference of highway officials of several Western States was held at Portland, Oreg., in February, 1930. It was then agreed to use the following specifications for road oils:

	Grade 0	Grade 1	Grade 2	Grade 3
Asphaltic residue of 80 penetration, per cent	30 to 40	50 to 60	63	95
Viscosity, Saybolt Purol at 122° F., seconds	15 to 40	40 to 80	200 to 300	--------
Float test at 50° C., seconds, minimum	--------	--------	--------	250
Float test at 25° C., seconds, maximum	--------	--------	200	--------
Flash point, Pensky-Martens, °F., minimum	150	175	225	395
Water and sediment, per cent, maximum	2	2	2	--------
Water, per cent, maximum	--------	--------	--------	½
Sediment, per cent, maximum	--------	--------	--------	1
Bitumen soluble in CS_2, per cent, minimum	99	99	99	99
Bitumen soluble in CCl_4, per cent, minimum	99.65	99.65	99.65	99.65
Bitumen insoluble in 86° Baumé paraffin naphtha, per cent	--------	--------	--------	6 to 27
Penetration of residue after loss on heating at 163° C. for 5 hours, minimum	--------	--------	--------	125

NOTE.—Grade 0 and 1 to be used for dust palliative and prime applications, grade 2 for mixing, and grade 3 in surface treatment, generally as binder material applied hot.

USE OF HEAVIER OILS DESIRABLE IN MIXED-IN-PLACE CONSTRUCTION

It has been evident for some time, particularly on roads carrying fairly heavy traffic and on those where moisture and climatic conditions are not entirely favorable, that the use of a bituminous binder heavier than the light oils now used would give better results

Typical Appearance of the Light-Oil-Mixed Roads of the West

in the mixed-in-place type of construction. With this in mind some experimental work using slow-drying emulsions and cut-back asphalts of the heaiver grades has recently been done in California.

EXPERIMENTS WITH SLOW-DRYING EMULSIONS SHOW PROMISE

The problem of developing an emulsion that would have a sufficiently retarded breaking point to permit mixing has not as yet been completely solved. Although material of this type is being made with a relatively slow break, experience shows that fine aggregate and dust accelerate the breaking of the emulsion. It has also been observed that the emulsion has a tendency to combine with the fines and leave the larger particles uncovered. This condition necessitates considerable rolling to incorporate these coarser particles and obtain a bonded mass.

Mixing an application of oil or water with the aggregate delays the breaking of the emulsion and makes it possible to obtain a better coating of the stone. Keeping the amount of fines in the aggregate to a minimum also reduces this difficulty. Although these recent changes in methods of construction and materials have in general improved the results obtained with this type of emulsion, further work along this line may well be done, as its use in the mixed-in-place type of treatment has not yet proved entirely satisfactory.

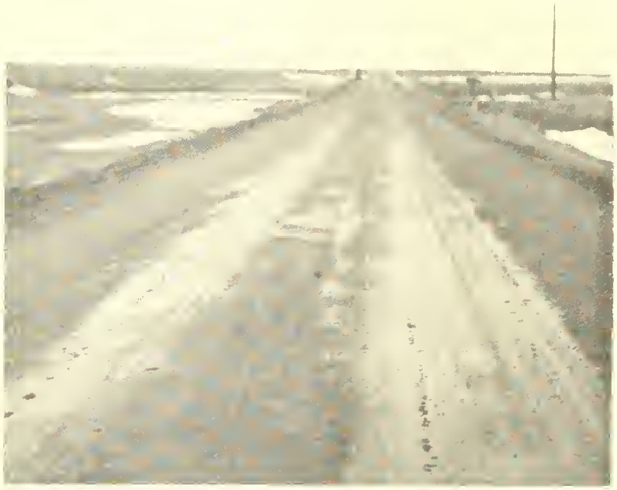
A Three-Inch Oil-Mix Road in Arizona. Early Failures Occurred Because the Aggregate Contained Caliche

CUT-BACK ASPHALTS ALSO USED SUCESSFULLY

Another material which has recently been tried out in mixed-in-place construction is cut-back asphalt. Both the 94 per cent grade of road oil and the 100 to 120 penetration asphalt material, cut back with kerosene, have been used. These materials seem to be entirely suitable for mixed-in-place construction. On the work so far completed the ease of construction and the excellent early appearance of the surfaces indicate that favorable results may be expected with this type of material. While its cost may be slightly higher than that of materials now in general use, it is believed that considerably better results may be obtained by using a heavier type of bitumen, as it offers greater resistance to raveling, particularly during freezing weather, and will withstand failure from moisture.

HEAVIER ROAD OILS NEEDED TO COMBAT WET WEATHER CONDITIONS

Experience has shown that oil-mixed surfaces in which the more fluid grades of road oil have been used fail rapidly under conditions of moist subgrade and wet weather. These road oils are not easily volatile under atmospheric conditions and stay in a fairly fluid condition in the surface mix. This fluidity and lack of great adhesiveness permit the water, moving up from the subgrade into the mix, to dislodge the oil surrounding the mineral aggregate in the lower strata, carrying the oil to the surface, where it gives the impression of bleeding or of an excessively rich mix. The water-logged surface loses its stability and begins to shove and rut, since the action of traffic under these conditions tends to destroy the bond of the mix. This effect is due, no doubt, to the scrubbing and kneading action of traffic, which causes dislodgement of the oil from the surfaces of the stone particles.



Typical Appearance of Bleeding on a Light Oil-Mixed Surface Caused Not by Excess Oil but by Moisture and Oil Rising to the Surface

Laboratory tests in conjunction with field observations indicate that road oils of high viscosity and high specific gravity resist the action of moisture better than the lighter materials, and also that a rich mix is more resistant to the action of moisture than a lean one. Although methods of construction or modifications of the materials may be devised for their more successful use, light oils are not at present proving satisfactory for mixed-in-place treatment under wet or freezing conditions. Consequently, there has arisen a great deal of interest in the use of heavier bituminous materials, such as cut-backs and emulsions, for this type of construction.

Table 2 gives the analyses of typical oils obtained from different fields and by different processes of manufacture used in the mixed-type of construction in the West and also the results of the Hubbard-Field stability tests on specimens molded from mixtures of these oils and sheet-asphalt sand.

TABLE 2.—*Test results on bituminous materials and Hubbard-Field stability values on sand mixtures*

B. P. R. No.	28825	28836	28855	30574	30941	31750	32591	32583	32584	32622	32553
Field	California	California	California	California	Texas		Wyoming	California	California	California	Wyoming
Process of manufacture	Topped residue	Topped residue	Topped residue	Topped residue		Cracking-plant residue	Cracking-plant residue	Cracking-plant residue	Cracking-plant residue	Fluxing	Fluxing
Type	70 to 80 road oil	70 to 80 road oil	70 to 75 road oil	60 to 70 road oil	No. 65 road oil	60 to 70 road oil	70 to 80 road oil	70 to 80 road oil	70 to 80 road oil	100 to 120 asphalt cement, cut back	94+ road oil, cut back
ANALYSIS											
Specific gravity, 25°/25° C	0.967	0.961	0.974	0.966	0.960	1.094	1.045	0.999	0.991	0.970	0.993
Flash point, ° C	107	143	99	68	85	163	189	144	122	79	105
Specific viscosity, 50° C., Engler	65.9	41.4	65.5	29.0	55.5	52.2	74.1	69.8	31.2	78.9	78.2
Loss, 163° C., 5 hours, 50 grams, per cent	8.64	5.48	9.82	15.42	13.46	4.83	1.92	3.65	5.58	13.28	13.13
Float test on residue, 50° C., seconds	32	23.8	36	37.4	44	22.8	43.4	54.8	39.6	75.8	91
Bitumen soluble in CS_2, per cent	99.95	99.91	99.94	99.92	99.88	98.91	99.63	99.80	99.77	99.79	99.97
Bitumen insoluble in 86° Baumé naphtha, per cent	9.89	7.50	12.21	13.18	19.27	26.03	21.73	13.96	13.68	8.05	17.04
Per cent residue of 100 penetration	70.14	66.05	67.15	62.7	65.9	69.5	74.4	73.7	72.1	75.3	78.0
Residue:											
Penetration at 25° C	85	88	103.5	104	113	103	98	93	113	103	98
Penetration at 0° C	12	15	19	22	38	13	16	17	21	19	26
Softening point, ° C	46.3	46.5	44.8	44.3	47	41.5	44.4	45.6	43.9	44.2	45
Ductility at 25° C., centimeters	110+	110+	110+	106.5	75	110+	110+	110+	110+	110+	110+
Ductility at 1.5° C., centimeters	6.8	5.2	11.9	7.0	6.0	6.2	(1)	4.7	5.0	7.0	5.5
Stability,[2] Hubbard-Field, at 25° C.:											
After 1 day at 60° C.—											
Per cent of bitumen volatilized	5.93	4.27	6.28	5.84	6.46	1.82	2.20	1.38	2.95	7.18	7.03
Stability, pounds	655	512	683	712	863	615	978	863	755	925	1,238
After 5 days at 60° C.—											
Per cent of bitumen volatilized	8.83	5.15	10.71	9.86	11.35	4.38	3.08	2.83	4.86	12.33	13.38
Stability, pounds	795	555	925	805	983	675	1,188	1,075	880	1,575	2,362
After 10 days at 60° C.—											
Per cent of bitumen volatilized	9.82	8.78	11.20	11.26	13.25	5.56	3.40	3.89	6.65	14.85	16.21
Stability, pounds	1,112	712	1,175	1,125	1,135	875	1,188	1,563	1,400	2,362	3,125

[1] Snapped.

[2] The cylinders for the stability test were made with Potomac River sand of the following grading:

Passing	10	20	30	40	50	80	100	200
Retained	20	30	40	50	80	100	200	---
Per cent	11.0	10.4	18.2	18.7	23.4	6.9	9.5	1.9

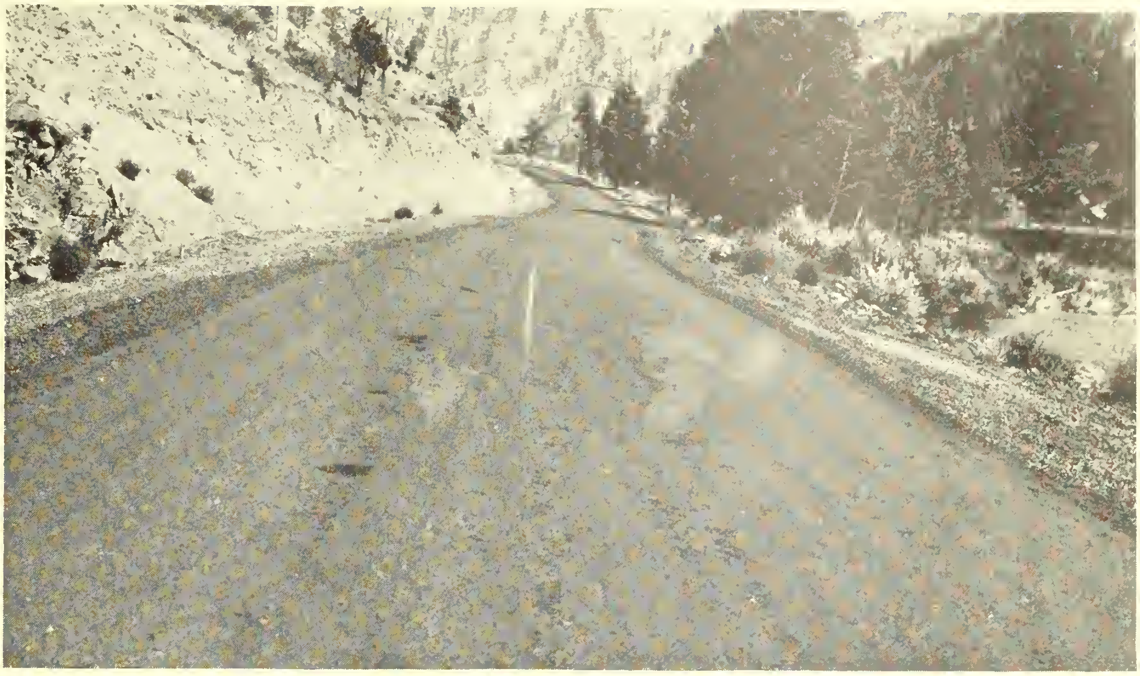
Eight per cent of bituminous material was used throughout. Materials were mixed and molded at room temperatures and subjected to a temperature of 60° C. in a Freas oven for the period stated. Specimens were tested for stability at 25° C.

CUT-BACK ASPHALTS SHOW RELATIVELY HIGH STABILITY

While there have been no recognized tests for determining the relative efficiency or binding power of the various oils that have been used in the oil processing, recent tests, of which those recorded in Table 2 are a part, have been made in the bureau's laboratory, using the Hubbard-Field stability test as a criterion. These tests indicate that there is a wide difference in the stability and the rate of hardening of these various road oils in fine-graded mixtures. The comparatively high stability obtained with the two cut-backs indicates that this type of material should prove more resistant to the destructive action of water, surface raveling, and displacement than the oils now in common use.

These two cut-backs represent a type of bituminous material which has only recently been used in the West, but which has been extensively used in the East for both surface treatment and cold mixing. In the Eastern States gasoline and similar distillates have generally been used as fluxing media. It was thought that the use of gasoline in cut-backs for mixed-in-place work would result in a too rapid stiffening of the road mix, thus preventing thorough covering of the stone and rendering the compaction of the finished surface more difficult. Therefore, after experimental work in the laboratory, two grades of cut-back, using a less volatile solvent, such as kerosene, were suggested and

FAILURE OFTEN OCCURS FROM RAVELING DURING THE DRY SEASON FOLLOWING THE ACTION OF MOISTURE ON THE MIX DURING THE WET SEASON



Early Raveling Develops When Insufficient Oil is Used in Construction. This Mix Contains About 25 Per Cent More Oil Than Called for by McKesson and Frickstad Formula

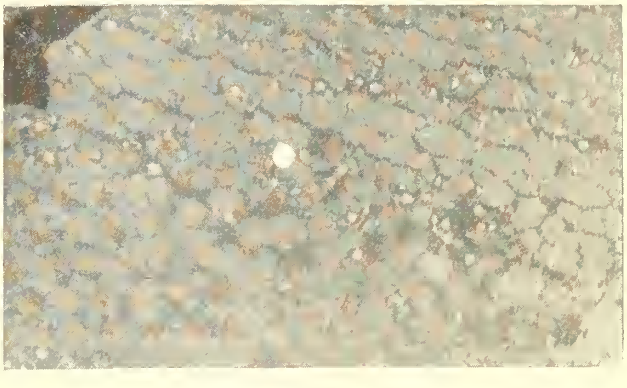
are being tried out in the West.[2] These two cutbacks are a 94+ per cent road oil, which is similar to a 150 to 200 penetration asphalt cement, and a 100 to 120 penetration oil asphalt. When cut back with 20 to 25 per cent of kerosene as the fluxing medium they give a product having a specific viscosity (Engler at 50° C.) of 70 to 80. The evaporation of the kerosene is not too rapid to interfere with the covering of the stone and subsequent compression, and the finished pavement develops a stability which could not be obtained with the usual light oil used in this type of construction.

PROPER AMOUNT OF OIL AN IMPORTANT CONSIDERATION

The amount of oil necessary for a particular job depends largely on the grading and character of the aggregate. The more fine material present, particularly of the size passing the 200-mesh sieve, the more oil is required. In all cases sufficient oil should be used to coat all the particles and partially fill the voids. The use of less than this amount results in early raveling and failure of the surface. Allowance must also be made if the aggregate is of the character that absorbs oil. This is particularly true when the aggregate contains an appreciable amount of soft limestone.

Several methods are in general use for determining the amount of oil required. The most common of these are the stain-test formulas, such as the McKesson and Frickstad formula and its modifications, and the "cut-and-try" method. The latter involves the determination of the requisite amount of oil on a short portion of the road as a guide in conjunction with the grading of the aggregate, determined periodically for the remainder of the project. Some engineers depend on the cut-and-try method, using the stain and formula, or both, as guides during the work. In any case, good judgment and considerable experience are necessary to determine the proper amount of oil to be used.

The tendency now is to use more oil than formerly, and considerably better results are being obtained. On the early work in many of the States the amount of oil used was determined by the stain tests, and the formula was developed primarily on the basis of experience in southern California. It has since been found that considerably more oil is required in areas with colder climates. In some of the States, and with certain types of mixture, it is frequently found necessary to use as much as 50 per cent more oil than is called for by the stain test or by the McKesson and Frickstad formula.[3]



Typical Failure of a Light-Oil-Mixed Surface During Dry Weather Following the Action of Moisture

Table 1, previously referred to, gives the amounts of oil actually used in a number of projects and the amounts which would have been used had the formula been followed.

T. E. Stanton, materials and research engineer of the California State Highway Commission, has recently developed a formula by which he believes the required percentage of oil can be accurately determined.

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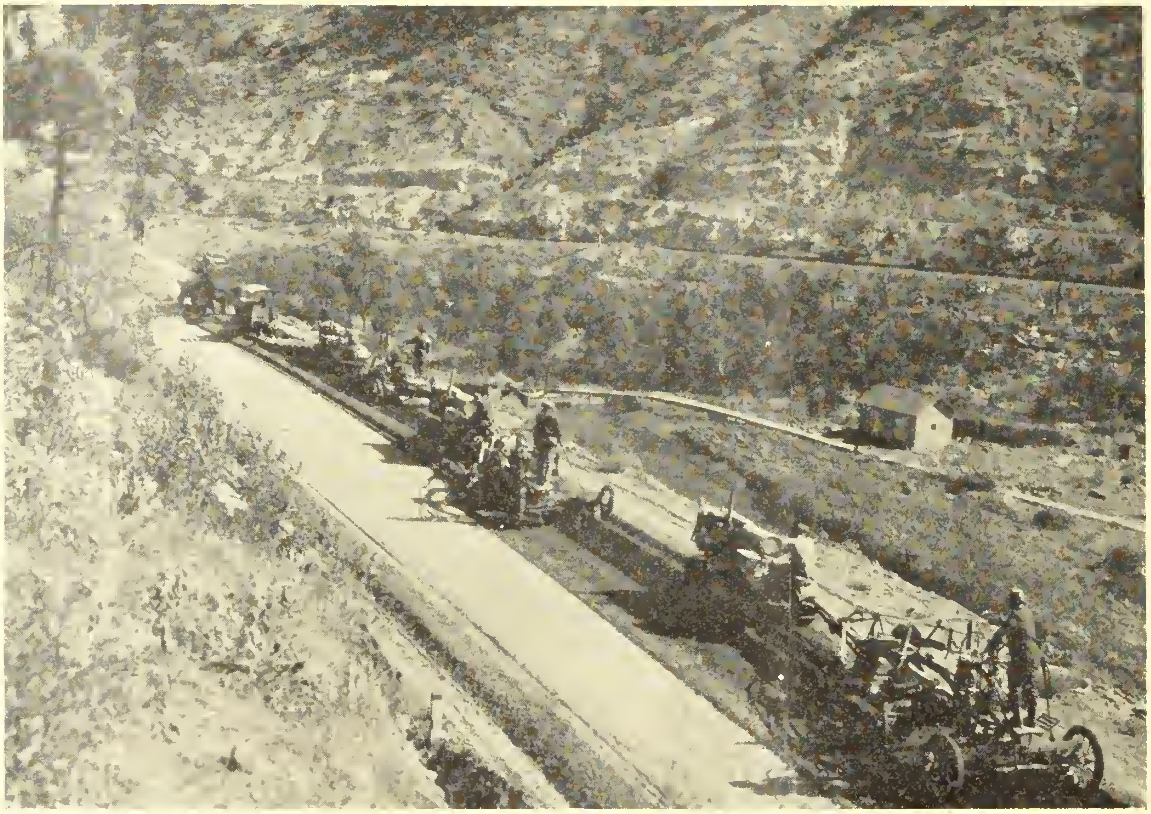
[2] Materials of this type were first used on the experimental road which extends westerly from the California-Nevada State line for a distance of about 10 miles along the Truckee River. This project was conducted cooperatively by the division of highways, California Department of Public Works, and the Bureau of Public Roads.

[3] $P=0.015a+0.03b+0.17c$, where P=the percentage of oil required, a=the percentage of aggregate retained on the 10-mesh sieve, b=the percentage of aggregate passing the 10-mesh and retained on the 200-mesh sieve, and c=the percentage passing the 200-mesh sieve.



Constructing a Mixed-in-Place Surface. Applying Bituminous Material, Disking, and Mixing

The formula is based on the surface area of the aggregate as determined by sieve analysis and modified by surface characteristics. For materials of average grading and average surface characteristics the following simplified formula is proposed for use in the field:

$$P = 0.020\,a + 0.045\,b + 0.180\,c$$

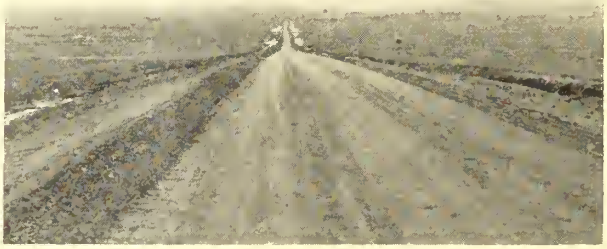
Where P = percentage of oil required, a = percentage retained on the 10-mesh sieve, b = percentage passing the 10-mesh and retained on the 200-mesh sieve, and c = percentage passing the 200-mesh sieve.

For very fine mixes, such as 100 per cent passing a ¼-inch sieve, it is recommended that the coefficient of c in this formula be changed to 0.150, whereas for coarse mixes, with only 50 per cent passing a ¼-inch sieve and only 4 to 5 per cent passing the 200-mesh sieve, the term should be increased to 0.20 c.

Another matter which should be considered in determining the amount of oil to be used is the moisture in the aggregate. If the moisture present, which is generally appreciable, is not taken into account, extractions and stain tests may indicate high oil content when in reality the mix is lean. An error in judgment may occur if the appearance of the mix is used as the guide in determining the amount of oil to use, as a small amount of moisture will invariably give the appearance of more oil than is actually present.

ONE-AND-ONE-HALF TO THREE INCH SECTIONS USED FOR MIXED-IN-PLACE SURFACES

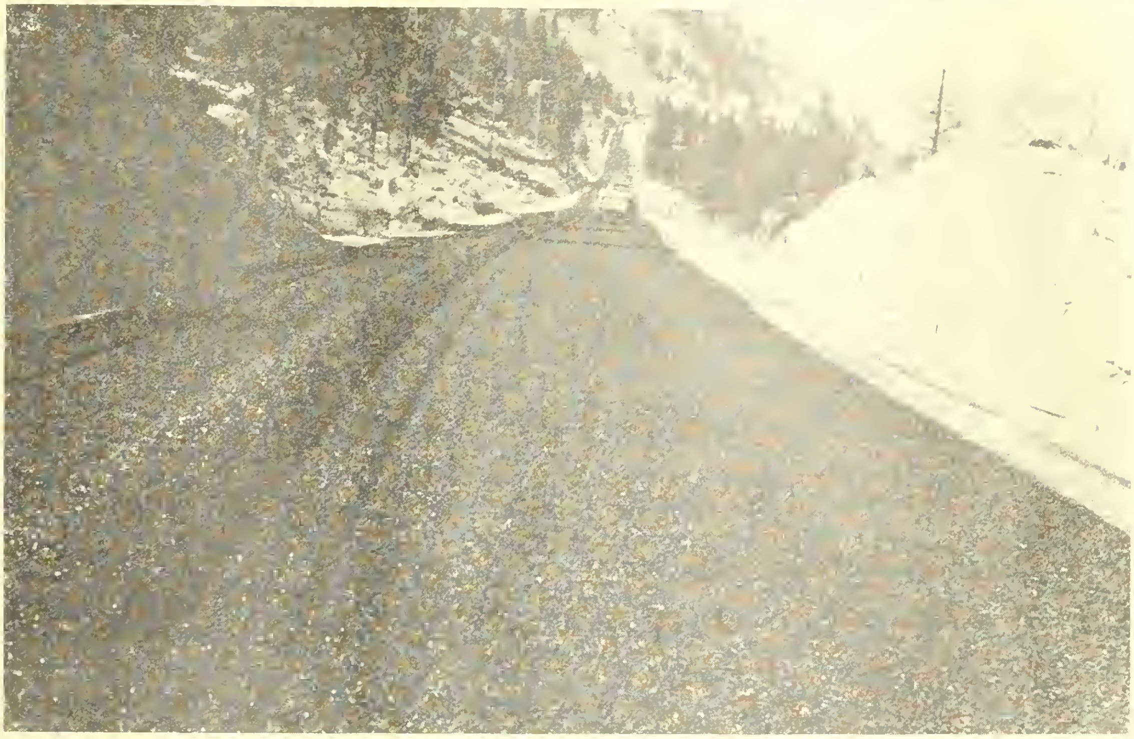
The thickness of oil-mixed surfaces has been found to range from 1½ to 3 inches, but in the majority of the cases examined it was 2 inches. Washington seems to be the only State using the 1½-inch section, and this may be explained by the fact that this type of construction is regarded in Washington as a temporary improvement, later to be strengthened by surface treatments or to be replaced by some higher type of construction. Washington also primes the base before constructing the mixed mat. This procedure no doubt improves the base condition and also serves to strengthen the wearing surface. With the exception of Washington, however, there appears to be a growing tendency to regard 3 inches compacted as the minimum thickness for a satisfactory mat.



Light-Oil-Mixed Surface Rapidly Failing on Account of Insufficient Oil. Note Coarse Aggregate Thrown to the Edges

VARIOUS MAINTENANCE METHODS USED

Maintenance methods used on oil-mixed surfaces depend not only on the condition of the road but also on the policy of the engineers in charge. The general practice is to use one or more of the following methods: (*a*) Repairing holes and breaks as they develop by using premixed material; (*b*) enriching the surface periodically by light applications of road oil; (*c*) scarifying and remixing with additional oil; and (*d*) applying surface treatments consisting of hot application materials or emulsions and covering with crushed stone or gravel.



100 to 120 Penetration Asphalt Cut Back with Kerosene. Smoothing of the Mix Prior to Compaction

The practice of surface treating the oil-mixed surface in order to provide greater wear resistance and to reduce the maintenance cost seems to be gaining in favor. This treatment provides an inexpensive method for improving oil-mixed roads to meet more adequately the needs of increased traffic.

The application of additional light oil to enrich the mix is used quite extensively in southern California, but does not seem to be much in favor elsewhere. The main objection to this treatment is the raveling and peeling which occurs under traffic during the period of drying. This method of maintenance seems to be of doubtful value except where traffic can be detoured for a time while the surface is hardening.

LIGHT OIL-MIXED SURFACES DEFECTIVE IN MOIST CLIMATES

Northern California, Oregon, and Washington favor surface treatment rather than the light-oil-mixed type, particularly in sections where wet subgrades or moist and freezing weather conditions prevail. It has been the experience of these States that light-oil-mixed surfaces do not prove satisfactory under these conditions. Water from the subgrade seems to rise freely to the surface, carrying with it the oil from the mix below. As this process continues the mix becomes unstable and rapidly rolls and displaces under traffic. The loss of stability seems to be aided by the action of the water on the light oil as this material appears to lose much of its body and adhesive qualities.

The early behavior of the mix when acted on by moisture gives the appearance of richness; that is, it bleeds and tends to shove and become wavy. After some time the surface mat breaks, raveling occurs, and failure soon develops. Until recently excess oil was believed to be responsible, and this misunderstanding tended to keep the quantity of oil used at a minimum.

The oil-mixed type of surface is far from being waterproof. The amount of oil used is generally sufficient only to coat the mineral aggregate without an excess to decrease the voids materially. Furthermore, the compaction is not sufficiently thorough. These two conditions render the pavement porous, with a tendency to become water-logged.

COST DATA DISCUSSED

Such data as are available indicate that the cost of the oil-mix treatment, exclusive of the cost of the aggregate, ranges from $1,000 to $2,000 per mile for an 18-foot width of 2 to 3 inches compacted depth. This figure does not include the cost of the base stone, which varies considerably, depending chiefly on the cost of the aggregate in place. For a large amount of this work under a range of conditions the cost for a 5 to 6 inch compacted crushed gravel or stone course ranges from $3,500 to $4,500 per mile.

Although definite figures on the cost of maintenance of oil-mixed roads are very limited, such data as are available indicate that the treatment is economical in comparison with untreated macadam.

The cost of surface maintenance of 600 miles of untreated macadam in Idaho during 1928 was $285 per mile, exclusive of material replacement, while on 200 miles of the oil-treated surface it was $294 per mile. The traffic on the former was 100 to 200 vehicles per day and on the latter from 100 to 1,200.

The annual maintenance cost in Wyoming during 1928, on a considerable mileage of treated roads, averaged $330 per mile.

The New Mexico Highway Journal for May, 1929, gives the following summary in proving the contention that oil-treated roads are cheaper than those which are untreated:

> Gravel and crushed stone roads in New Mexico constitute 1,450 miles. Approximately 1,000 miles now require 4 inches of material to restore them to their original thickness. It has been found that the untreated roads lose approximately 1 inch of material per year.

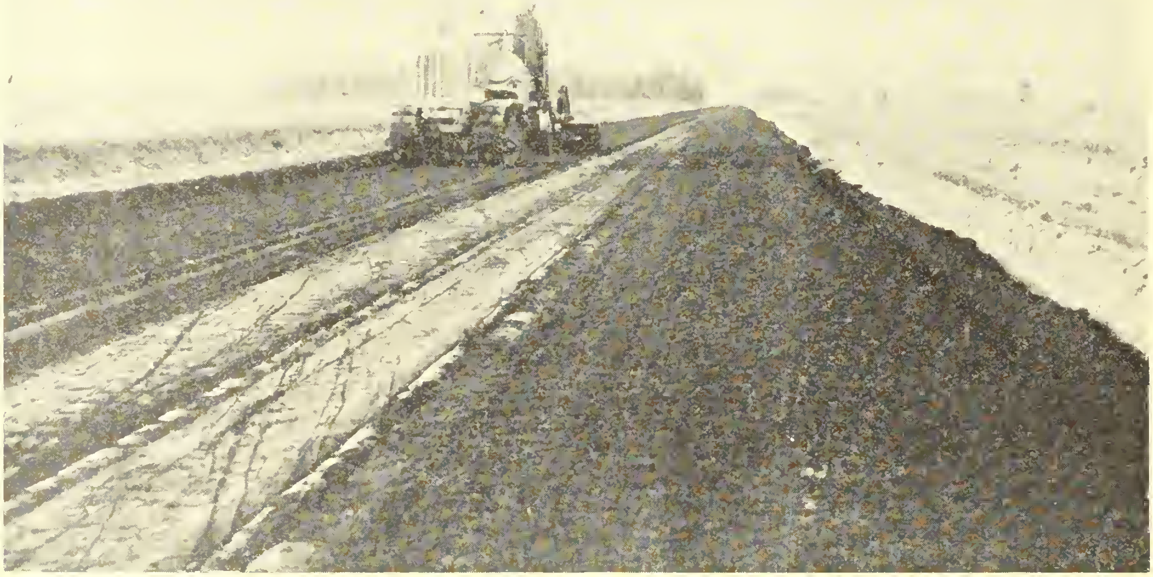
	Untreated	Oiled
Original cost per mile of construction	$2,860	$3,800
Annual maintenance cost includes:		
Interest on investment, at 6 per cent.	171	228
Replacement of material lost	715	----------
Maintenance	350	250
Total annual cost	1,236	478

CONCLUSIONS

The results of this survey lead to the following conclusions regarding the use of the mixed-in-place method with gravel or stone aggregate.

1. In sections where conditions are suitable, such as arid and semiarid regions, the light-oil-mixed treatment is proving satisfactory and economical and will undoubtedly continue to be used extensively.
2. Light oil does not give a sufficiently well bonded surface to prevent raveling under heavy traffic, particularly in cold weather, nor does it withstand the action of moisture.
3. For unfavorable conditions a heavier asphaltic oil, or a soft asphalt cement cut back with a distillate, such as kerosene, naphtha, a combination of both, or a suitable asphalt mixing emulsion, should be used. A surface treatment using a hot application material or emulsion on either the light-oil mix or on the untreated macadam may be used to obtain more satisfactory results.
4. An insufficient amount of oil has been used on much of the work up to this time, resulting in many failures and high maintenance cost. The fact that a bituminous mix has the greatest resistance to deterioration when its bitumen content is the maximum allowable, as judged from the stability required, seems to have been ignored generally on bituminous work in the West. It seems, indeed, that the attempt has been made to use the minimum amount possible.
5. Very little shoving and corrugating occurs. When such defects are found they are generally due not to excess oil but to moisture or subgrade conditions.

OIL TREATMENT OF EARTH AND SANDY SOILS



94 Per Cent Road Oil Cut Back with Kerosene Mixed in Place with Blow-Sand Soil

"BLOW-SAND" ROADS TREATED IN WYOMING AND NEBRASKA

Oil treatment of earth and sandy soils by the mixing method has not been tried to any large extent in the Western States. Some work of this type has recently been done in Wyoming and Nebraska. The soil in both cases is a very fine grained and poorly graded material commonly called blow sand.

The method of construction consists of mixing in two layers to produce a final compacted depth of about 4 inches. A light road oil was used on the work in Wyoming, while in Nebraska both a light road oil of 60 to 70 per cent asphalt and a heavier material of 94 per cent asphalt and 100 to 120 penetration asphalt cut back with a slow drying distillate of the kerosene type were used.[4]

Sufficient time has not elapsed since this construction to allow conclusions to be drawn regarding its success. The sandy soil is very unstable, and for that reason the light oil may not add sufficient stability. By the use of the heavier material much greater bond is obtained, as well as a much better road surface.

The following table gives the typical grading of the soils which were oil mixed in Wyoming and Nebraska:

	Wyoming Project 198	Nebraska Project 313A
	Per cent by weight	*Per cent by weight*
Passing 10-mesh sieve	99.7	100.0
Passing 20-mesh sieve		98.8
Passing 30-mesh sieve		96.8
Passing 40-mesh sieve	93.8	87.7
Passing 50-mesh sieve		65.9
Passing 80-mesh sieve	75.9	21.4
Passing 100-mesh sieve		10.9
Passing 200-mesh sieve	19.8	1.4

The amount of oil used in this construction was about 7 per cent on the Wyoming work and 7 to 8 per cent on the Nebraska job. As shown by the grading of the soil, the Wyoming material should require more oil, because of the large amount of material passing the 200-mesh sieve. This is borne out by the early behavior, which shows the surface tending to ravel and dissipate under traffic. In this case additional oil will no doubt have to be added and mixed into the surface.

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[4] During the summers of 1929 and 1930 the division of bridges and highways, Nebraska Department of Public Works, and the Bureau of Public Roads conducted cooperative experiments, using these materials in the treatment of road surfaces composed of fine sandy soils.

SOIL ROADS TREATED IN CALIFORNIA

Treatment of soil roads with light oils, similar to those used on crushed stone and gravel roads, by a method which is a combination of penetration and mixing has been used on a large scale in Fresno County and other counties of California and has proved highly satisfactory. These soils vary from those that are almost all sand to a red loam. Although the more sandy material has given the best results, oil treatment of the more plastic material has also been successful. In general, where the soil lacks granular material, sand or gravel is first mixed into the surface.

The following table gives the analysis of a typical soil on which these treatments are made:

	Per cent by weight
Retained on 10-mesh sieve	0.0
Passing 10-mesh, retained on 20-mesh	.1
Passing 20-mesh, retained on 30-mesh	2.5
Passing 30-mesh, retained on 40-mesh	4.3
Passing 40-mesh, retained on 50-mesh	9.0
Passing 50-mesh, retained on 100-mesh	37.0
Passing 100-mesh, retained on 200-mesh	23.3
Passing 200-mesh	23.8

METHOD OF CONSTRUCTION DESCRIBED

Progressive construction covering a period of about two years is required in order to build a satisfactory oil-surfaced soil road. The construction treatments, four or five in number, are applied during the summer months of the first two years, the procedure being as follows:

The road surface is bladed to the cross section desired, material for about 1 inch of soil being left in windrows at the edges. A light oil of about 60 to 70 per cent asphalt content is then applied at the rate of three-fourths gallon per square yard and covered with the soil bladed in from the windrows. As soon as the traffic begins to break through the first treatment the surface is scarified to a depth of 2 or 3 inches and a second treatment similar to the first is applied. Similar treatments are added during the following year until the oiled mat has sufficient thickness and richness to carry the traffic, after which only light treatments are required every two or three years.

An Oil-Treated Road in Fresno County, Calif.

More than 2,000 miles of these roads have been built in Fresno County alone. They are in excellent condition and have the appearance of sheet asphalt. Many of them carry traffic in excess of 2,000 vehicles per day.

The good behavior of these roads is probably due largely to the dry and warm climate and to the fact that the soil is largely sandy. In Illinois, where there has been considerable oiling of the soil roads, the results have not been so favorable, probably because of a combination of conditions such as moisture in the soil, the moist and cold climate, and less granular and well-graded soil.

SURFACE TREATMENT

THE fact that the heavier bituminous materials used in surface-treatment work are only slightly affected by moisture and that this treatment forms a nonporous surface mat which largely prevents the escape of the air under the surfacing, and thereby tends to prevent the rise of moisture from the subgrade, is probably responsible for the good behavior of this type.

Light Blading of Cover on Surface-Treatment Work Used Quite Generally in the West

The present attempt to use heavy asphaltic oils or soft asphalt cements of both the cut-back and emulsion types is motivated by the desire to obtain not only a more stable and wear-resistant surface but also one that will resist the action of moisture.

Surface treatments are used not only on crushed-stone and crushed-gravel macadams but also on old and new oil-mixed surfaces. Several types of surface treatment differing primarily only in thickness and methods of construction are used, depending largely on the surface to be treated.

"ARMOR COAT" TREATMENT USED IN CALIFORNIA

In California the so-called "armor coat" is used extensively. It is constructed essentially in accordance with the following outline:

1. Thoroughly sweep the old surface and prime with one-half gallon per square yard of 50 to 60 fuel oil.
2. Apply hot about one-eighth to one-fifth gallon per square yard of 95 per cent grade road oil.
3. Cover with three-fourth to one-half inch crushed stone or gravel at the rate of 50 to 60 pounds per square yard.
4. Blade and roll lightly.
5. Apply about three-eighths gallon per square yard of the same grade of road oil.
6. Cover with one-half to one-eighth inch crushed stone or gravel at the rate of about 20 to 30 pounds per square yard.
7. Roll lightly.

On some of the older work a prime application of 60 to 70 road oil was used. This material was not entirely satisfactory, as it hardened very slowly, during which time the surface picked up greatly under traffic. A prime application is not so necessary on a well-bonded oil mix or on some other types of bituminous surfacing, but should be used on an untreated macadam to tie the treatment to the base, as well as to harden and bond it. A fairly hard asphalt cement cut back with a distillate of the kerosene type which penetrates well and is fairly quick drying, or a fluid residual from certain types of cracking-plant process, should prove highly satisfactory for this purpose.

Light tar of 8 to 13 viscosity is being used extensively in the East, but is not available in the West.

VARIOUS PRODUCTS USED FOR SURFACE TREATMENTS

Emulsions made of heavy road oils and high-penetration asphalt cements are coming into rather extensive use on surface treatment work. This is true not only in California but in Oregon and Washington as well.

In Oregon the practice in surface treating is to use a prime application of light oil or gas-oil tar. Otherwise, the light type of surface treatment used is quite similar to the "armor coat" treatment used in California.

Equipment Used in Southern California for Long-Haul Delivery of Oil

"MULTIPLE LIFT" METHOD USED IN OREGON

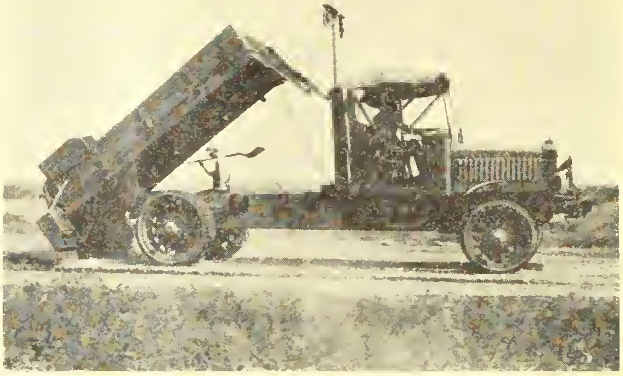
Oregon has more recently been using what is termed a "multiple lift" type of surface treatment when a thickness greater than that obtained by the common treatment is desired. A thickness of 2½ inches or more can be obtained by this method. One advantage of this process is that the thickness of the surface mat can be varied to suit the traffic and base conditions on a particular project. The steps involved and the materials used are as follows:

1. Thoroughly sweep the existing road surface.
2. Apply a tack coat of about 0.15 gallon per square yard of 95 per cent road oil, hot.
3. Cover with 50 to 60 pounds per square yard of 1½ to ¾ inch crushed aggregate.
4. Smooth by blading lightly and roll.
5. Apply about one-third gallon per square yard of the same type oil.
6. Cover with about 30 pounds per square yard of three-fourth to one-half inch crushed aggregate.
7. Smooth as before and roll.
8. Apply about one-fourth gallon per square yard of the same oil.
9. Cover with about 30 pounds per square yard of one-half to one-eighth inch crushed aggregate.
10. Smooth as before and roll.

Sometimes the final treatments, items 8, 9, and 10, are repeated to give a better sealed surface. Greater thickness is obtained by increasing the number of "lifts"; that is, by doubling or trebling the treatments given under items 3, 4, and 5.

The cost of a 2½-inch mat constructed in this manner is rather high. This fact is due to some extent to the many steps involved in the construction. To obtain the same thickness of mat, a mixed-in-place method of construction would be more practical and probably more economical. However, the heavy bituminous material would have to be designed for cold application and mixing.

METHOD INVOLVING PARTIAL EMULSIFICATION USED IN OREGON



Detachable Spreader Used in Applying Cover Material on Surface-Treatment Work

Oregon has recently devised a method of partially emulsifying the hot oil for surface-treatment work, particularly for treatments using heavy oil. The plan is to combine water and an emulsifying agent, such as soap, with the hot oil at the instant it strikes the stone. This is accomplished by means of a second spray bar attached so that a spray of water is directed against the oil spray. About 10 per cent of water is required. The use of this method results, it is said, in better covering and penetration of the aggregate.

WASHINGTON PRACTICE DISCUSSED

In Washington surface treatment is used not only on the crushed gravel and stone macadams but to a large extent on such oil-mixed surfaces as are built. The treatment is similar to the light treatments used in Oregon and California, with the exception that two successive prime applications of light oil are applied. Here, as in other sections where light oil has been used for priming the base, considerable peeling and raveling has occurred under traffic during the hardening of this material. This difficulty was overcome in many instances by mixing the excess primer with fine material bladed in from the shoulders. Light blading of this material over the surface of the roadway has corrected many of the irregularities prior to the application of the heavier surface-treatment oil.

Surface-Treated Gravel Macadam Road near Crater Lake, Oreg.

The riding qualities of the surface-treated roads in general compare favorably with those of the mixed-in-

(Continued on p. 208.)

THE MOST RECENT METHODS ADOPTED FOR THE USE OF TAR, BITUMEN, AND ASPHALT IN ROAD CONSTRUCTION

REPORT OF AMERICAN ENGINEERS TO THE SIXTH INTERNATIONAL ROAD CONGRESS

By A. W. DEAN, Chief Engineer, Massachusetts Department of Public Works; R. H BALDOCK, Maintenance Engineer, State Highway Commission of Oregon; G. H. HENDERSON, Chief Engineer, State Board of Public Roads of Rhode Island; PREVOST HUBBARD, Chemical Engineer, The Asphalt Institute; and CHARLES H. MOOREFIELD, State Highway Engineer, South Carolina

INTRODUCTION

BECAUSE the title assigned to this report contains terms which are not expressed in exact conformity with common practice in the United States, a brief statement of the significance of these and other similar terms from the American standpoint should be made in order to prevent misunderstanding on the part of representatives from other countries.

Bitumen as here used is a generic term, including that portion of tar, petroleum, and asphalt products which is soluble in carbon disulphide.

The adjective "bituminous" is equivalent to the expressions "containing bitumen" or "with bitumen." Thus, a bituminous mixture is a mixture containing bitumen in the form of tar, petroleum, or asphalt, and a bituminous treatment is a treatment made with bitumen in the form of tar, petroleum, or asphalt.

Tar is a bituminous product which yields pitch when partially evaporated or fractionally distilled, and is condensed during the destructive distillation of organic materials, such as coal, oil, lignite, peat, wood, etc. The great bulk of tar used for highway purposes in the United States is obtained from the destructive distillation of coal, although a portion is produced by the destructive distillation or cracking of gas oils in the manufacture of carbureted water gas.

Asphalt is a semisolid or solid cementitious material inherently black to dark brown in which the predominating constituents are natural bitumens occurring as semisolids or solids in nature or obtained by refining petroleum.

Rock asphalt is rock naturally impregnated with asphalt.

Asphaltic road oils are liquid products of various degrees of viscosity containing asphalt which may be recovered by evaporation or distillation.

Cut-back asphalts are semisolid or solid asphalts dissolved or rendered fluid by admixture with petroleum distillates, which are frequently but not necessarily of a highly volatile nature.

BITUMINOUS TREATMENT OF EARTH ROADS

The two widely different types of natural soil roads, namely, clay and sand, are susceptible to entirely different types of treatment with bituminous materials. As might be expected, intermediate types of a sand-clay nature are susceptible to a variation in treatment between the two extremes.

CLAY ROADS

Where the subgrade and surface of an existing soil road is a plastic clay, conditions are least favorable for obtaining permanently satisfactory results with bituminous treatment, and too much should not be expected from such treatment. Nevertheless, where a suitable type of bituminous material has been used judiciously on clay roads a distinct improvement in comfort of travel and in traffic-carrying capacity during adverse weather conditions has been effected at a reasonable cost. Such work is perhaps best typified by the large mileage of bituminous-treated clay roads in the States of Illinois and Minnesota.

During dry weather these roads, while capable of carrying heavy loads, become exceedingly dusty, and during long spells of wet weather, particularly in the winter and early spring, the natural soil becomes so soft and slippery as to either eliminate traffic or restrict it to light loads and create considerable hazard. Because the natural soil is highly absorptive and retentive of water, the bituminous surface first constructed is apt to break up during protracted spells of wet weather, and in order to effect any permanent improvement repeated surface treatments are required, with the expectation that as the old surfaces become broken up by traffic and ground into the underlying soil a bituminous soil mixture of appreciable thickness is gradually produced which is more water resistant and stable than the natural soil. Such retreatments are usually made in the spring, and frequently additional treatments are given in the fall. The thicker the layer of soil and oil mixture eventually produced the greater becomes the load-supporting value of the road during unfavorable weather conditions.

General experience has indicated that in order to obtain progressive improvement in the condition of clay roads by bituminous surface treatment the oil should possess high penetrating nonemulsifying characteristics but need not necessarily develop highly cementitious qualities after application. Asphalt which may be present in the oil should develop as such rather slowly, so that sufficient opportunity is afforded for thorough blending of the oil with the soil before the mixture sets up. For this reason, what are known as slow-curing oils are generally preferred. Application of such material is usually made by means of a pressure distributor after the road has been properly shaped with a road machine during compaction under traffic. The oil is first heated to between 150° and 190° F. (66° and 88° C.) usually by means of a jet of live steam. The oil as applied, therefore, often contains a small amount of water, which is not considered objectionable. Single applications are usually made at a rate of not over one-half gallon per square yard when the surface is fairly dry. A slightly moist condition is, however, to be preferred to a dusty bone-dry condition, which tends to make the oil ball up and not distribute itself uniformly. The slow-curing highly penetrative oil is absorbed by the soil to such an extent that no mineral cover is ordinarily required. From two to seven days may be required to secure thorough absorption, during which time it is advisible to eliminate traffic from the treated surface. If for any reason the surface is tacky and likely to pick up when traffic is admitted, it should be covered with a very light layer of soil bladed in from the sides of the road.

In certain cases a fairly viscous asphaltic oil is purposely used to produce a tacky surface, which is then covered with a light coating of gravel or stone chips similar to the stabilizing course later described for sand-clay treatment.

By treatment with oil alone the road is made practically dustless until the bituminous surface becomes cut up under traffic during wet weather, and the resistance of the road to rapid deterioration in wet weather is somewhat increased.

It has been found that repeated annual or semiannual treatments of the same nature work additional improvements, and for this reason attempts have been made with some degree of success to hasten such results by means of a number of treatments applied at relatively close intervals, each treatment but the last being harrowed into the soil and mixed in place with a road blader by methods similar to those later described in connection with mixed-in-place gravel roads. Thorough mixing of oil with the fine clay soil by this means is, however, rather difficult to secure, and the mixed-in-place method has not proved as promising for clay as for other types of road material.

SAND ROADS

Where the natural soil is practically pure sand its method of treatment with bituminous material will to a large extent depend upon the grading and character of the sand grains. If it is of such nature that it would prove satisfactory for use in sheet-asphalt construction it is also well adapted for use in the construction of what has been termed "the sand-asphalt road."

The sand-asphalt road is the ultimate development of bituminous-surface-treated and mixed-in-place oil-sand roads, made possible by the excellent quality of the sand itself. In this class of work, which has been extensively constructed in Massachusetts, Delaware, and North Carolina, a premixed base or foundation is first constructed with a mixture composed of approximately 93 per cent of the natural sand and a 7 per cent paving grade of asphalt cement, heated and combined at a paving plant such as is used in the preparation of sheet-asphalt mixtures. No mineral filler is ordinarily used in the foundation mixture. In North Carolina the compacted base usually has a thickness of 3 inches and is laid between temporary or permanent plank forms which have been staked in place prior to preparation of the subgrade. After thorough compaction by rolling the surface of the base is given a squeegee coat of hot asphalt cement and then covered with a 2-inch thick wearing course of sheet-asphalt paving mixture made with the same sand, to which mineral filler is added prior to combining it with the asphalt cement. Where natural drainage conditions are good this type of pavement has proved quite satisfactory under reasonably heavy traffic.

In the Cape Cod region of Massachusetts excellent results have been obtained with repeated bituminous-surface treatment of natural sand soil roads. The modern methods employed in this work call for use during the first two years of a road oil which yields approximately 45 per cent of asphalt by the evaporation test. One-third gallon per square yard of this oil is applied the first year and this treatment repeated during the second year. During both years the road is bladed or dragged at frequent intervals to maintain a smooth, uniform cross section. At the end of the second year a bituminous carpet has been produced which is usually quite hard. During the following year a more highly asphaltic oil containing 60 per cent asphalt is applied at the same rate per square yard, and during the fourth and fifth years an 85 per cent asphalt oil is used. The fifth treatment is made at a rate of one-fourth gallon per square yard. At the end of five years the bituminous surface has been built up to a point where it no longer requires annual treatment.

Some sand soils are not suited to either of the types of treatment above described, and first require modification by admixture with clay or clay soils before treatment.

SAND-CLAY ROADS

Sand-clay roads, of which thousands of miles have been constructed, particularly in the southeastern section of the United States, are composed of natural or artificial mixtures of sand and clay in such proportion as to produce a wearing course vastly superior to either all clay or all sand. Such mixtures usually consist of from 65 to 85 per cent of sand, from 9 to 25 per cent of clay, and from 5 to 20 per cent of silt. Topsoil is a natural mixture of sand and clay found in surface deposits and is frequently used for this purpose just as it is stripped from the fields. While vastly superior to the average graded earth road, an untreated sand-clay or topsoil road becomes softened during prolonged wet spells and is very dusty in dry weather.

During the past few years considerable attention has been directed to means of improving such roads by surface treatment with bituminous materials so as to create a better year-round wearing surface and increase their traffic-carrying capacity. It has been found that in order to secure satisfactory results from such treatment the existing road surface requires careful advance prepartion, which consists of shaping and compacting where necessary to produce a true well-bonded surface. In Florida a thin stabilizing layer of fine pea gravel is sometimes worked into the surface during such preparation. The treatment itself consists of the following steps:

1. Thorough sweeping to remove all loose particles and dust, in so far as possible, without disturbing the bonded surface.

2. Application of a prime coat of tar or asphaltic oil of sufficiently low viscosity to penetrate the upper portion of the old road. This priming material is applied at the rate of from one-fifth to one-third gallon per square yard, depending upon the absorptive capacity of the road surface, and is intended to harden and moisture-proof the soil and provide a surface to which the heavier asphaltic or tar product later applied will adhere.

3. Application of an asphaltic or tar product of relatively high viscosity in sufficient amount to retain a later application of fairly coarse mineral aggregate. This material, which is frequently asphalt cement of between 150 and 200 penetration, is applied by means of a pressure distributor at a temperature of 250° to 325° F. (121° to 163° C.) and at a rate of from 0.3 to 0.5 gallon per square yard, depending upon the surface texture of the primed base and the size of mineral aggregate used for cover.

4. Application of mineral aggregate cover consisting of broken stone or slag passing a 1-inch or 1¼-inch screen, which, after brooming and rolling, combines with the second application of bituminous material to produce a thin wearing course knitted to and overlying the old sand-clay road which thereafter serves as base.

5. Where winter conditions are likely to be severe an additional light application of cut-back asphalt may be applied and covered with fine mineral aggregate to thoroughly seal the surface if after a few months' traffic the bituminous mat previously constructed does not possess the desired degree of waterproofing. This practice is regularly followed in South Carolina, the seal coat being applied before the first winter of service.

The serviceability of the bituminous-surface treatment above described is dependent upon careful and intelligent maintenance applied promptly as required. The patrol system of maintenance is almost essential, as the surface should be under constant observation to detect and correct defects as they develop and prevent deterioration of adjacent areas due to progressive disintegration which takes place very rapidly if the original trouble is not promptly eliminated. It has been found that retreatment of bituminous surfaces on this type of road will be required at intervals of two, three, or four years, and may consist of a light seal or enriching treatment or of a heavier treatment to thicken or smooth up the original wearing course. South Carolina makes a practice of using the first-mentioned type of retreatment on old treatments that are still smooth, but uses a mixed-in-place treatment with as little as one-fourth gallon of bituminous material and 25 pounds of crushed stone per square yard on old treatments that have become roughened. In South Carolina and Florida many bituminous-surface-treated sand-clay roads which have been in service for three or four years are successfully carrying traffic of considerably over 1,000 vehicles per day and in some instances in excess of 2,000 vehicles per day.

SURFACE-MIXED GRAVEL AND BROKEN-STONE ROADS

The extraordinary easy-riding quality of a well-constructed and properly maintained gravel road has been widely noted and commented upon by engineers as well as the traveling public. Most of these gravel roads are at their best when the gravel is moist but not saturated with water. If they could be kept permanently in this condition their traffic-carrying capacity would be much higher and the cost of maintenance would be greatly reduced. The same has been noted of what have been termed "traffic-bound broken-stone roads," composed of small-size broken-stone fragments, together with fines produced at the crushing plant, and constructed in the same manner as gravel roads, being compacted by traffic accompanied by frequent blading with a road machine. For best results the proportion of coarse to fine fragments should be such as to produce a dense aggregate when compacted.

Although perhaps not generally recognized, the basic principle of the use of bituminous material in what has been termed the "surface-mixed" or "mixed-in-place" gravel and broken-stone road appears to be the substitution of bituminous material for that amount of water which would produce the most satisfactory condition for the particular untreated aggregate. This class of construction has been most extensively adopted by California, but other States, including Oregon, Idaho Arizona, New Mexico, and Nevada, have closely followed the California method in less extensive work, and modifications of this method have been used to some extent in Wisconsin and Minnesota.

In the treatment of an existing traffic-bound gravel or broken-stone road sufficiently thick to eliminate the necessity of using additional mineral aggregate the method of treatment as practiced in California is briefly as follows:

The old road is first scarified to a uniform depth of from 3 to 4 inches and then harrowed, after which an asphaltic oil of from 60 to 70 per cent asphalt content is applied to half the width of the road at a temperature of from 150° to 200° F. by means of a pressure distributor, the rate of application being approximately 1½ gallons per square yard. The treated surface is immediately harrowed to prevent the oil from being picked up by the mixing equipment. Power or tractor-drawn graders are used for mixing by starting the grader at the center and working the bituminous-treated aggregate in a windrow, first to the edge and then back to the center of the road. This operation is continued until the mixture is of uniform color and texture, after which it is bladed into place. The other half of the road is then treated and mixed in the same manner, after which compaction is secured under traffic during blading in order to maintain a smooth, even surface. After compaction the road has the general appearance of a well-bonded asphaltic concrete pavement, although the mixture is not as hard and firmly bonded as in the case of a hot-mixed asphaltic concrete. Maintenance consists of blading with a road machine similar to the maintenance of an ordinary gravel road, light scarifying being sometimes necessary to maintain a smooth surface. In some cases the oil-mixed road is given a surface treatment of bituminous material when it is in good condition, in the same manner and with the same class of product as would be used in the surface-treatment water-bound macadam.

Analyses of samples taken from surface-mixed roadways show that wide variation in grading of mineral aggregate may be tolerated so long as the correct amount of bituminous material is used for the particular aggregate. As a guide to determining when the proper amount of bituminous material has been used, that portion of the treated aggregate which passes a sieve having 2-millimeter openings is subjected to a stain test. California has found that mixtures which produce a light yellowish brown stain in which the impression of the individual sand particles may be distinguished and which is not blurred or blotched give the most satisfactory results. Where too little oil is used the mixture is apt to ravel under traffic and the use of much excess is apt to cause distortion under traffic. In the former case the remedy is to apply additional oil, which is harrowed and mixed into the scarified road. If an excess of oil has been used the trouble may be corrected by scarifying the road and adding additional aggregate of the same character as originally used.

Idaho, under more severe climatic conditions, has found it desirable to incorporate a somewhat greater proportion of oil than has California and frequently uses as much as a gallon and a half of oil per square yard to a 2-inch thickness of mineral aggregate. Idaho's experience has indicated that the surfacing material which is to be treated should be limited to a maximum size of 1 inch and should carry not less than 45 per cent of material passing the ¼-inch screen in the case of screened gravel and not less than 55 per cent of the smaller size for crushed gravel or crushed rock. Aggregates containing as high as 70 to 80 per cent of this fine aggregate have been satisfactorily treated.

In the case of new construction or when a surface-mixed wearing course is to be laid on an old gravel road, good practice calls for purchasing the mineral aggregate in two sizes, with the ¼-inch screen as the dividing line. These sizes may then be combined on the road in any desired proportions.

In some cases the mixture of gravel or broken stone with the asphaltic oil has been made at a paving plant and then laid on an oil road or prepared foundation. If this is done the bituminous-treated aggregate can best be placed by means of road machines and should be compacted by traffic, accompanied by frequent blading. The use of a roller for compaction does not give the same satisfactory results as in the case of the hot-mixed asphaltic concrete, and traffic compaction is therefore greatly to be preferred. The apparent advantage of machine mixing over mixing in place would seem to be insurance of greater uniformity of mix, but service results so far have not definitely indicated that the difference is sufficiently great to justify any material increase in cost due to plant mixing.

BITUMINOUS TREATMENT OF WATER-BOUND SURFACES

Ordinary gravel and water-bound macadam roads have been successfully surface treated with a great variety of bituminous materials, including nonasphaltic distillate, dust-laying oils, fluid and viscous residual tars and asphaltic road oils, cut-back asphalts, emulsified asphalts, and soft asphalt cements. All of these products are ordinarily applied by means of a pressure distributor, those of relatively high viscosity being preheated to such degree of fluidity as to permit securing the desired uniformity of application. For purposes of dust laying only the very fluid products which can be distributed cold are applied at a rate barely more than sufficient to saturate the dust particles on the road surface. Such applications have to be made at rather frequent intervals.

For the purpose of producing an impervious bituminous mat or carpet on the road surface, either fluid cut-back products, emulsified asphalts, or viscous residual products are used. Such work may be classified under single and double treatments, depending upon whether one or two treatments are used to produce the original mat surface. While the former has been most widely employed, it is believed that the double-surface treatment is ordinarily productive of a more durable mat, which justifies the slightly higher cost.

An unusual method of developing an emulsified asphalt application is used in Oregon, consisting of the installation of a separate water tank and spray bar on the pressure distributor, so that the asphalt and water with soap solution are distributed together from two separate spray bars on one distributor.

DOUBLE-SURFACE TREATMENT

Double-surface treatment usually involves use of a light fluid tar or road oil primer, preferably followed by application of a soft asphalt cement of from 150 to 300 penetration. It is particularly applicable to surfaces from which the dust can not be completely removed prior to treatment, as the priming material saturates such dust particles, penetrates the upper portion of the road, and puts it in condition which permits the heavier asphaltic product to adhere in much the same manner as has been described for the surface treatment of sand-clay roads. The priming material should be applied only in sufficient quantity to be absorbed by the road surface within from 24 to 48 hours, so that it will not pick up under traffic. It should not be covered with mineral aggregate, but should receive directly the treatment with soft asphalt cement or very heavy asphaltic oil. The former is preferable from the standpoint of its better ability to retain the coarse aggregate which is applied for final cover. For this purpose, ¾-inch stone, free from dust, should be used and thoroughly compacted by rolling. If dust is present it is apt to sift over the asphalt-treated surface and prevent the large stones from adhering firmly.

Surface treatments in general will not make an old rough surface smooth, and it is therefore necessary to bring the road to a true, uniform contour before applying the treatment as described. This is ordinarily done before the priming material is applied, but excellent results have been obtained in the State of Washington by means of two applications of priming material, the first made as previously described and the second consisting of a 60 to 70 per cent asphaltic road oil applied at the rate of 0.2 gallon per square yard, which is then covered with crushed stone or gravel from one-half inch to dust, applied at the rate of 50 to 60 cubic yards per mile. This treatment is then worked over the road with a grader so as to true up the surface with a mixture of the aggregate and asphaltic oil. The heavier asphaltic product, consisting of a 95 per cent asphaltic road oil, is then applied at the rate of 0.35 gallon per square yard and covered with three-fourth to one-eighth inch mineral aggregate.

Where the old road is very rough a retread treatment, such as later described, is to be preferred unless the old road is first scarified and reshaped.

SINGLE-SURFACE TREATMENT

In the single-surface treatment of a previously untreated water-bound surface the bituminous material is expected to perform two functions, namely, to penetrate the surface to some extent and at the same time to hold a substantial cover of mineral aggregate. If a road oil is sufficiently fluid to serve the first purpose it does not possess sufficient viscosity or body to perform the second as satisfactorily as does a soft asphalt cement, and the mineral aggregate cover is therefore likely to be displaced by traffic and pick up and spatter vehicles for some time after treatment. Residual bituminous products which behave in this manner are, unfortunately, being quite widely used, with the result that for a matter of several days and often a week after treatment the road is in an unsatisfactory condition from the standpoint of the user.

The use of a soft asphalt cement for single treatment of previously untreated water-bound surfaces is also unsatisfactory, because it does not penetrate and attach itself to the surface, which, if at all dusty, is apt to produce a condition which will allow traffic to completely remove the mat over sections of the road area and deposit it on other sections, which then become soft and mushy. The resulting road is apt to be very rough and uneven.

Probably the most satisfactory class of materials for single-surface treatment are cut-back products of sufficient fluidity to penetrate the road surface and containing a distillate which is not so readily volatile as to leave the bituminous base before penetration is secured. Emulsified asphalts have also given good results in single bituminous treatments where the original surface is not excessively dusty.

Dust is perhaps the greatest enemy to successful surface treatments, and good practice calls for its removal, so far as possible, by thorough sweeping prior to application of bituminous material.

RETREAD TREATMENT

Retread construction has been adopted with a number of variations in different parts of the country, particularly in connection with the treatment of old gravel or macadam surfaces where something more durable than surface treatment is required at a lower cost than would be involved in the construction of a bituminous-macadam wearing course.

In Pennsylvania and a number of other States a mixed-in-place method has been extensively used with excellent results. In this method the old road is first scarified, reshaped, and consolidated with the addition of new stone or gravel if necessary to increase its thickness. If, however, the old road is of suitable thickness and of uniform cross section it is merely swept to remove all excess dust and loose material. After preparation of the old road a layer of crushed stone of from 1¼ to ¾ inch diameter is spread to a depth of 2 inches and is then treated with refined tar or cut-back asphalt at the rate of from one-half to three-fourths gallon per square yard. The applied material is then worked back and forth with a blader until it has become thoroughly mixed and worked to a true, uniform cross section, and has begun to set. This may require from four to five days. At the end of this period a seal coat of the bituminous material is applied at the rate of one-fourth gallon per square yard, and lightly covered with stone chips. Traffic is permitted to operate continually during the entire construction. Compaction will, of course, be hastened by use of a roller immediately following the blader, rolling being continued until after the seal coat of bituminous material has been applied and covered with stone chips.

The State of Indiana has developed a radically different method under the name of "retread" which, in certain respects, is quite similar to asphalt macadam, but uses less bituminous material. The old road is prepared to serve as foundation in the same manner as previously described, and upon the base is spread 2 to 1¼ inch stone for a depth of from 2 to 3 inches, loose, as desired. This course is lightly rolled and then treated with about 0.4 gallon per square yard of a cut-back asphalt containing a fairly slow drying distillate. The surface is then bladed with the blade so set that only the upper portion of the treated stone is moved and only the bitumen-coated stone pushed in front of the blade. After blading to proper cross section the road is rolled once with a 10-ton roller, and after being subjected to traffic for a few days is then rolled very thoroughly. The texture of the surface at this stage is rather open, and application of one-half to five-eithths inch stone chips is therefore made in sufficient quantity to fill the surface voids. These chips should be clean and free from dust and should be used only in sufficient quantity to accomplish the purpose of filling surface voids. The second application of the cut-back asphalt is then applied at the rate of 0.2 gallon per square yard and the bitumen-coated chips bladed or dragged with a sled drag, after which they are rolled into the surface. Some two weeks later a third application of the cut-back asphalt is made at the rate of approximately 0.15 gallon per square yard, covered sparingly with one-half inch stone chips, and the road finished off by thorough rolling. The total quantity of bituminous material, amounting to three-fourths gallon per square yard, is less than half the quantity of bituminous cement used in the construction of a bituminous-macadam pavement of the same thickness, but excellent results have been secured by this method where carefully constructed and carefully maintained. Maintenance should start immediately after completion of the surface, and in some cases it has been found desirable to reseal the surface after a period of one or two years, using from one-fourth to one-half gallon per square yard of cut back asphalt and 30 to 40 pounds per square yard of ¾-inch stone chips.

Because of the fact that the retread method uses a small quantity of bituminous cement and does not tend to shove under traffic, it is possible to gradually build up or reinforce an old road by successive retread treatments at suitable intervals.

SURFACE TREATMENT OF BITUMINOUS MACADAM, ASPHALTIC CONCRETE, BRICK, AND PORTLAND CEMENT CONCRETE

For bituminous-surface treatment of pavements higher than the water-bound type a rapid-drying cut-back asphalt containing a highly volatile distillate is most commonly used, application being made by means of a pressure distributor. By using a cut-back or emulsified asphalt it is possible to control distribution so as to obtain a very thin carpet, which is particularly desirable in the treatment of old asphalt macadam or asphaltic concrete pavements from which the seal coat has been worn away but which otherwise are in good condition. One-half-inch stone or slag chips make an excellent cover material for such treatment and are generally to be preferred to sand. When the old surface is at all rough, however, a retread or mixed-in-place treatment with the same materials is preferred. In certain cases, where an old sheet-asphalt pavement is so badly cracked as to require seal coat, a relatively fine sand may be used to advantage over a very light squeegee coat of the cut-back asphalt, the advantage of the sand in this case being that it helps to fill the cracks in the underlying surface.

The construction of a thin bituminous mat on old brick and concrete roads, while occasionally successful, has not proved sufficiently dependable to warrant its very extensive adoption, and such surfaces are usually covered with an asphalt wearing course of asphalt macadam, asphaltic concrete, or sheet asphalt, constructed in the usual manner after truing up the old surface with binder course or asphaltic concrete mixture which is frequently prepared with a cut-back or emulsified asphalt and laid cold.

BITUMINOUS MACADAM

Bituminous macadam pavements have perhaps reached a higher degree of perfection in the States of Massachusetts and Rhode Island than elsewhere, although excellent pavements of this type are constructed in other sections of the country. The bituminous macadam, while one of the simplest types to construct, is more susceptible to abuse from careless workmanship or use of improper methods than are most of the other types of bituminous pavements, for the reason that if almost every principle of construction is violated it is still possible to produce a pavement of reasonably good appearance immediately after completion, in which the defects will not develop until after it has been subjected to traffic for a considerable period of time. When properly constructed the pavement has a long life even under heavy traffic conditions, as has been repeatedly demonstrated by the experience of the two States above-mentioned.

Where the bituminous macadam is to be constructed upon a gravel or broken stone foundation, particular attention should be paid to remedying poor subgrade conditions. In Rhode Island it has been the practice to lay a blanket course of sand or fine gravel from 4 to 6 inches in depth over areas of bad mucky soil prior to constructing the foundation course of coarse gravel or stone varying in size from 8 inches downward. Thorough consolidation of newly constructed foundations prior to laying the base course and wearing course is essential to securing a lasting uniformity of contour of the finished pavement, and it has been found advisable to allow such foundations to season under traffic prior to laying the wearing course.

The construction of adequate shoulders and side supports is also of vital importance, as the edges of the wearing course are more susceptible to displacement and deterioration than any other portion of the roadway. Constructing the foundation wider than the wearing course is also desirable, in order that loads transmitted from the surface at the sides may be more evenly distributed over the subgrade.

The construction of the pavement calls for considerable care in certain details. Hard trap rock is the preferred mineral aggregate, ranging from 2½ to 1½ inches in size. In any event the individual size of fragments should be reasonably uniform, so as to uniformly distribute voids in the course to be penetrated. A larger size should be used for the softer varieties of rock. After spreading and initial compaction the surface should be carefully examined to discover segregated areas containing an excess of fine material, and in such areas the material should be removed and replaced with suitable aggregate.

The asphalt cement, emulsified asphalt, or refined tar used for first application should be applied uniformly over the surface, preferably by means of a pressure distributor. A battery of as many as three large pressure distributors has been successfully used in the construction of wide roads, so that the whole width of the surface may be treated at one time. In such case one distributor spreads along one side of the road over one-third its width and is followed by the second distributor, which is so operated as to make a neat joint with the first application. The third distributor follows the second in like manner and carries the application to the other side of the road. The bituminous material should, of course, be applied at such temperature that uniform penetration of the broken-stone course is secured. If for any reason during original application uncovered streaks are produced, such places should be covered by means of hand pouring pots prior to application of the keystone.

Just sufficient keystone should be spread over the first application to permit rolling. After initial rolling enough additional keystone is added to fill the surface voids. Hand or machine operated brooms are of distinct value in spreading the keystone uniformly and are used to remove all excess prior to application of the bituminous seal coat. The keystone should be free from dust and of approximately ¾-inch average diameter. Asphalt makes the most durable seal coat and is applied by means of a pressure distributor at the rate of approximately one-half gallon per square yard. This seal coat is covered with clean stone chips thoroughly rolled into the surface. The total amount of bituminous material used in the construction of the most durable asphalt macadam pavements averages approximately 1 gallon per square yard for each inch of thickness. When an asphalt cement is used for seal coat on a properly constructed bituminous macadam, maintenance is reduced to a minimum and retreatment or replacement of seal coat should not be required for many years.

Some engineers who have specialized in this type of construction do not believe it possible to overroll a bituminous macadam after the bituminous material has been applied, and place such stress upon obtaining ultimate compaction as to require continuous backrolling for several days after the pavement is completed.

HOT-MIXED PAVEMENTS

Types.—The three principal types of hot-mixed asphalt pavements are: Coarse aggregate asphaltic concrete, fine aggregate asphaltic concrete, and sheet asphalt. The asphaltic concretes are ordinarily laid to a finished thickness of 2 inches over a substantial foundation, and a coarse aggregate asphaltic concrete, without mineral filler, is extensively used in the construction of foundations from 3 to 6 inches thick, commonly known as "black base." Sheet asphalt is laid to a finished thickness of 3 inches, including a 1½-inch binder course and a 1½-inch wearing course. If a total thickness of 2½ inches of sheet asphalt is desired, the wearing course and not the binder should be reduced to 1-inch thickness. The design and construction of hot-mixed asphalt pavements have been so thoroughly covered in paving literature that it seems unnecessary in this report to attempt to discuss many of the details which are of recognized importance in securing satisfactory results.

The method of preparation, laying, and compacting of all three types is essentially the same, although the composition varies, as shown below.

1. Coarse aggregate asphaltic concrete consists of a mixture of broken stone, broken slag, or gravel (usually passing a 1¼-inch screen and retained on a ¼-inch screen), sand, mineral filler, and asphalt cement. The coarse aggregate predominates and usually runs to from 55 to 65 per cent of the total mixture. Asphalt varies from 5 to 8 per cent, and the remaining 30 to 40 per cent consists of sand and mineral filler.
2. Fine aggregate asphaltic concrete is essentially a sheet-asphalt mixture containing from 25 to 35 per cent of ½ to ¼ inch stone chips and from 7 to 9½ per cent asphalt cement.
3. Sheet-asphalt binder course mixture is a coarse aggregate asphaltic concrete containing no mineral filler and as a rule carries from 4 to 5½ per cent asphalt cement, the aggregate being so graded as to produce a reasonably dense mixture but not carrying such a high percentage of fines as to create a smooth sealed surface when compacted. Sheet-asphalt wearing course mixture is composed of sand, mineral filler, such as limestone dust passing a sieve of 200 mesh, and asphalt cement. The character of the individual constituents and the proportions in which they are combined should be so controlled as to produce a mixture which, when compacted, will contain not over 5 per cent and preferably not less than 2 per cent voids and at the same time possess sufficient stability to prevent shoving or displacement under traffic.

STABILITY OF MIXTURES

During the past few years considerable attention has been devoted to developing a physical test which will measure the stability or resistance to displacement of compressed sheet-asphalt paving mixtures, and one of these methods, known as the stability test, is coming to be quite extensively used as an aid to design and control of such mixtures. Many interesting facts have been learned from the stability test which may be frequently applied to pavement work as a substitute for the old arbitrary specifications and rule-of-thumb methods.

Thus it has been found that between two sands of identically the same grading and general appearance under the microscope one may be capable of producing mixtures of high stability, while the other may produce mixtures of relatively low stability only. This is evidence that in the past the importance of exact grading limitations has been greatly overestimated. It has been found that, starting with a given sand, the stability of the mixture is increased by the addition of successive increments of mineral filler up to a certain critical point, after which further addition of filler produces a decrease in stability. The point of maximum stability for combinations of any given sand and mineral filler will approximate the point at which the further addition of filler begins to increase the percentage of voids in the total aggregate.

It has been found that considerable difference in stabilizing and void-filling characteristics exists between different fillers, depending not only upon their relative fineness but upon their general composition and surface texture. It has been shown that the addition of asphalt to the total aggregate, in quantity more than sufficient to fill the voids, will greatly lower the stability of the mixture. From the standpoint of resistance to water action, it is undesirable to design a mixture with an aggregate containing an extremely low percentage of voids, because in such mixtures the film of asphalt is too thin, and where possible the mixture should therefore be designed to carry at least 9½ or 10 per cent of asphalt.

It has been found that a variation of 10 points penetration in the asphalt cement produces but little effect upon the stability of the mixture as compared with normal variation in percentage and character of mineral filler and normal variations in character of sand. With some aggregates a variation of as much as 1 per cent of asphalt may produce five times as great an effect upon the stability of the mixture as will a 10-point variation in penetration of the asphalt.

PAVING PLANT AND CONSTRUCTION DETAILS

Uniformity of plant output is essential to high-class construction, and perhaps the most important detail to observe in this connection is to insure thorough premixing of the constituents of the dry aggregate before adding the asphalt cement. Unless this principle is put into practice, no amount of mixing after the asphalt has been added to the aggregate will secure the desired uniformity of output.

The use of a drum mixer of several tons capacity has been found to produce just as satisfactory mixtures as can be made by the pugmill type. In connection with the latter it has sometimes been found advantageous to change the blades so as to force the mixture around the circumference of the mixer rather than to toss it from the ends toward the center.

No matter how carefully a mixture is designed, it will not develop its inherent stability unless it is thoroughly compacted. Thorough compaction is therefore the first fundamental in connection with construction. While the same degree of compression can probably be obtained by rollers of varying design and weight, modern practice tends to favor use of a heavy 3-wheel roller for initial compaction of both sheet asphalt and asphaltic concrete paving mixtures, because maximum uniform compression is most easily obtained by this type of roller. In the case of sheet asphalt or fine aggregate asphaltic concrete, final rolling should be performed with a tandem roller in order to obtain a smooth uniform surface.

Smoothness or easy-riding quality of a newly finished asphalt pavement is almost, if not quite, as important as thorough compaction, and an increasing amount of attention is being paid to this detail. Finishing machines similar to those used in Portland cement concrete construction are being successfully used in the construction of hot-mixed-asphalt pavements in different sections of the country, and the State of California now requires the use of a finishing machine in its standard specifications. It seems quite probable that such machines will be very widely used in the future.

Spreading boxes for uniformly spreading bituminous-pavement mixtures or aggregates which are to be treated with bituminous materials have been used with such excellent results in some localities as to give promise of much wider adoption in the future. Spreading devices attached to the rear end of trucks, which have been developed in connection with the construction of State highways in Washington and Oregon, have proved especially satisfactory for applying thin layers of fine material aggregates over seal coats or surface treatments. When so used the trucks are slowly backed over the treated surface, the run-out from the rear end supplying sufficient cover to prevent the truck wheels from picking up the bituminous material on the road surface.

ASPHALT-BLOCK PAVEMENTS

Asphalt blocks are composed of a fine aggregate asphaltic concrete and are made in a variety of sizes, the one most generally used for paving highways being 12 inches long, 5 inches wide, and 2 inches deep. Where weight is an important consideration, as in the case of bridge floors, a smaller block, 8 inches long, 4 inches wide, and 1½ inches deep, is sometimes used. Asphalt blocks are usually laid upon a Portland cement mortar bed spread over the foundation, and it is of course important that the mortar be struck off so as to produce a uniform contour, true to grade and of uniform density. The asphalt blocks should be laid immediately upon the moist mortar, with close joints and uniform top surface. The blocks should be laid so as to make the lateral joints as tight as possible and the longitudinal joints broken by a lap of approximately 4 inches.

As soon as practicable after laying the blocks and making the necessary corrections the surface of the pavement should be swept clean and hot asphalt applied with a squeegee or machine distributor over the entire area. This is a rather recent innovation, as most of the older asphalt-block pavements were constructed merely with a sand filler. The application of asphalt is covered with a clean layer of thin coarse sand or stone screenings passing a ¼-inch screen before traffic is admitted.

On grades too steep for the ordinary smooth surface, asphalt blocks should be laid with recessed joints, thereby affording protection against slipping of horses and skidding of motor vehicles. In such cases the foundation and mortar bed are laid in the usual manner. The asphalt blocks are then laid upon the mortar bed one row at a time, with a ½-inch wooden strip between each course. The wooden strips are next removed and the joints filled with grout composed of 1 part Portland cement to 1½ parts of sand. Before the grout has taken its initial set it is scraped out of the transverse joints by means of a special tool to a depth of about one-half inch. This results in the formation of recessed joints one-half inch in width and about one-half inch in depth.

ROCK ASPHALT

The two principal classes of rock asphalt, namely, bituminous limestones and bituminous sandstones, are used in considerable quantities in the construction and maintenance of pavements. The former is usually enriched by the addition of an asphaltic flux and laid and compressed while hot, according to the usual methods for laying hot-mixed asphalt paving mixtures.

The bituminous sandstone, most of which is obtained from Kentucky, is, on the other hand, almost invariably laid cold. This product as marketed usually carries from 5½ to 9½ per cent of a soft bitumen coating a sharp grain silica sand, with very little, if any, extremely fine mineral particles present. When used in original construction it is usually laid to a compacted depth of 1½ inches. The product is crushed to pass a ¾-inch screen, with 80 per cent or more passing a ¼-inch screen. Where the bituminous limestone is to be cold laid it is crushed to pass a ¼-inch screen, and after compression is seal coated with material passing a ⅛-inch screen. When laid on gravel or broken stone foundations a bituminous primer should preferably be applied to the foundation surface, as described for other types of construction. The cold-laid rock asphalt can best be compressed during warm weather. In cold weather it is very difficult to secure satisfactory compaction. Cold-laid rock asphalt is frequently used in patching old bituminous pavements, for which purpose it shows up to best advantage.

COLD-MIX AND COLD-LAID BITUMINOUS PAVING MIXTURES

Cut-back and emulsified asphalts and cut-back refined tars have beeen extensively used in the preparation of bituminous concrete mixtures without preheating the aggregate. Such mixtures have been satisfactorily used for many years for the maintenance of bituminous pavements under the designation of "cold-patch mixtures" and are laid and compressed without preheating. In some cases it is advisable to cure the mixture by exposure to the air in piles for 24 to 48 hours before use in order to allow most of the volatile constituents to evaporate. These mixtures have also been used to some extent in the construction of cold-laid pavements.

There are a number of patented or proprietary mixtures which are prepared with cold or preheated sand with the idea of making a synthetic product of characteristics similar to the natural bituminous sandstones but of more uniform composition. These mixtures have been used for maintenance and construction in the same manner as described for rock asphalt. The asphalt used as a binding material in such mixtures is usually of softer consistency than would ordinarily be used in the preparation of a mixture containing similar aggregate which is to be laid hot. An interesting modification of this type which is intended to duplicate sheet asphalt is prepared by mixing a sheet-asphalt aggregate composed of sand and mineral filler with hard pulverized asphalt and a flux at such temperature that the hard asphalt will not actually be fluxed during the mixing process. Such mixture can be transported and handled cold, but is heated, immediately prior to laying, to the usual temperature for sheet-asphalt mixtures, at which temperature the fluxing process becomes complete. By this method it is claimed that an asphalt plant can be kept in continuous operation throughout the year, as against the usual 90-day period, the material being prepared and stocked for later transportation and use. Moreover, the contractor laying the pavement does not have to use as expensive equipment as though he were making the complete mixture.

Another somewhat similar process involves the incorporation of a hard pulverized asphalt with any of the usual mineral aggregates for asphaltic concrete or sheet-asphalt construction and then mixing same with a flux below the usual fluxing temperature. Such mixtures are laid without preheating, the fluxing operation gradually taking place after the material has been laid and compacted.

Some of the products above described may be considered as still in the experimental stage of development. As a general proposition, no fine aggregate mixture can be laid and compressed cold to secure the same desired density as required for a hot-laid mixture, unless the bituminous binder is of such soft consistency as not to immediately produce the firm bond secured in a hot-laid mixture. This is particularly true of the fine aggregate mixtures.

Perhaps the most extensively used and successful type of cold-laid mixture is produced by first coating a dry aggregate, consisting of commercial broken stone, with a petroleum distillate, such as kerosene, and then mixing it with hot asphalt cement of about the consistency used in bituminous-macadam construction. No attempt is made to use a graded aggregate for this purpose, as it would be difficult to spread and compress. Such products are made with aggregates of at least two different sizes to produce two mixtures which are laid separately. The coarser mixture is laid so as to duplicate quite closely the first course of a bituminous macadam, the advantage being that assurance is obtained that all of the individual stone fragments are uniformly coated with a relatively thick film of asphalt. After compression a much finer aggregate mixture is spread and compacted on the first course to produce a seal coat. Many pavements of this type have been in service for a considerable period of time and have proved satisfactory under fairly heavy traffic.

(Continued from p. 199.)

place types. The use of mechanical spreaders in applying the cover material and the method of smoothing the cover with blade graders before rolling have not only reduced the cost of construction but have greatly improved the riding qualities as well.

RESULTS OF SURVEY SHOW IMPORTANCE OF LOW-COST CONSTRUCTION

This general survey has shown that the bituminous treatments of low-cost roads, such as crushed gravel, crushed stone, and soil roads, are in general proving satisfactory, and that this work represents a large portion of the construction carried on in the Western States. Limited funds with great mileages prohibit to a large extent the construction of any of the so-called permanent and more expensive types. As this condition will probably continue for many years, it is reasonable to expect that in the future these States will continue to be engaged largely in the construction and improvement of the types now being built and in the development of such other relatively low-cost types as may prove suitable.

There is no objection on the part of the Western States to some increase in the cost of construction, provided such increase results in a decreased cost of maintenance. This is particularly true of those States which depend largely upon Federal-aid funds. Since the Government does not aid in maintenance, it is becoming more important, as the amount of road building increases, to keep the cost of maintenance, rather than that of construction as low as possible. To accomplish this end it can be expected that greater attention will be given to the use of types, methods, and materials best adapted to meet given conditions.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D. The Results of Physical Tests of Road-Building Rock. 15c.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
1216D. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No. 55. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
*93M. Direct Production Costs of Broken Stone. 25c.
109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATIONS

No. 76. The Results of Physical Tests of Road-Building Rock.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *914Y. Highways and Highway Transportation.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Connecticut.

Report of a Survey of Transportation on the State Highway System of Ohio.

Report of a Survey of Transportation on the State Highways of Vermont.

Report of a Survey of Transportation on the State Highways of New Hampshire.

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio.

Report of a Survey of Transportation on the State Highways of Pennsylvania.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

NOVEMBER 30,1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total	Estimated total cost	Federal aid allotted	MILEAGE Initial	MILEAGE Stage[1]	MILEAGE Total		
Alabama	2,178.0	$2,024,368.94	$963,400.88	70.9	26.8	97.7	$2,680,852.64	$1,318,156.05	120.4	4.2	124.6	$6,282,541.96	Alabama
Arizona	887.9	3,623,889.75	2,636,686.93	111.5	130.8	242.3	777,954.06	768,765.79	39.7	11.7	61.4	2,399,874.82	Arizona
Arkansas	1,785.8	6,679,858.69	3,086,243.22	197.0	50.4	247.4	1,128,328.37	668,744.83	23.4	13.5	38.9	2,780,139.73	Arkansas
California	1,930.9	7,909,854.10	3,383,099.64	192.6	14.0	206.6	1,825,079.50	899,558.46	2.6	18.0	21.6	4,427,520.40	California
Colorado	1,242.2	4,693,681.85	2,509,007.61	162.4	75.7	238.1	635,746.03	367,034.94	26.6	24.1	50.6	3,850,185.52	Colorado
Connecticut	246.9	3,117,621.20	1,302,014.40	19.8		19.8	456,213.95	84,060.00	6.8		5.6	1,282,645.45	Connecticut
Delaware	289.6	697,391.10	348,695.54	33.8		33.8	61,045.10	30,622.55	3.8		3.6	818,546.89	Delaware
Florida	633.8	4,680,361.00	2,229,234.31	76.0	6.6	81.6	638,794.08	254,135.15	21.9		21.9	2,640,709.47	Florida
Georgia	2,648.9	6,644,688.56	3,177,029.04	169.9	123.9	293.6	2,041,872.80	916,920.42	61.7	37.9	89.6	4,673,135.82	Georgia
Idaho	1,282.1	2,208,013.74	1,346,265.26	163.7	11.6	185.3	353,001.64	210,600.00	27.9	6.2	36.1	1,844,898.98	Idaho
Illinois	2,112.3	22,135,550.69	10,056,260.68	831.6	68.6	700.2	1,367,755.88	645,586.15	47.8		47.6	7,135,946.20	Illinois
Indiana	1,669.1	2,332,076.25	1,133,362.56	76.3		75.3	3,638,840.80	1,246,135.86	88.7		88.7	4,441,544.58	Indiana
Iowa	3,230.4	106,290.11	1,892.77		4.0	4.0	799,906.20	303,600.00		26.6	26.6	3,039,343.56	Iowa
Kansas	2,931.6	4,742,460.00	2,225,663.70	253.4	44.8	298.2	963,226.16	480,341.44	77.9	42.9	120.8	4,121,606.67	Kansas
Kentucky	1,417.3	6,787,619.35	2,915,375.56	136.3	178.9	313.2	3,380,207.33	777,679.89	63.2	35.6	98.6	1,569,730.32	Kentucky
Louisiana	1,397.3	4,875,971.76	2,264,239.69	172.0	20.9	182.9	542,276.46	236,764.18	15.0	7.1	22.1	2,621,358.23	Louisiana
Maine	680.4	2,482,503.65	832,271.08	54.0		54.0	678,620.39	285,808.09	.9		.9	1,818,144.20	Maine
Maryland	656.3	1,851,370.52	881,913.20	33.7	18.9	60.6	188,899.75	41,957.10		2.4	2.4	999,144.32	Maryland
Massachusetts	711.8	5,301,078.22	1,250,618.86	42.8		42.8	1,278,711.03	544,323.81	10.3		10.3	2,846,608.09	Massachusetts
Michigan	1,739.9	9,499,427.33	3,864,213.55	231.4	18.1	249.6	1,338,790.47	609,275.00	44.0	6.4	50.4	4,352,565.43	Michigan
Minnesota	4,210.1	6,128,606.07	1,790,416.62	80.9	137.1	216.0	1,418,187.96	546,745.64	6.2	40.4	46.6	2,747,399.60	Minnesota
Mississippi	1,840.2	694,023.67	296,715.35	35.7	7.7	43.4	422,696.83	208,805.55		20.3	20.3	5,510,836.47	Mississippi
Missouri	2,622.6	10,069,679.30	3,436,611.36	154.9	76.0	230.9	1,986,479.46	618,718.36	60.3	3.1	63.4	3,301,109.06	Missouri
Montana	1,873.9	7,675,698.38	4,332,660.68	589.3	44.1	633.4	895,210.00	486,922.85	98.0	11.5	109.6	4,708,614.16	Montana
Nebraska	3,817.6	6,119,229.71	2,439,276.31	269.6	97.2	368.7	633,065.12	187,203.65	23.2	14.1	37.3	4,143,529.85	Nebraska
Nevada	1,252.0	1,121,084.75	674,209.01	39.8	89.3	129.1	103,231.27	81,720.96	12.6	.8	13.4	1,987,644.63	Nevada
New Hampshire	378.9	1,217,789.47	429,124.66	21.1	1.6	22.7						672,280.18	New Hampshire
New Jersey	548.2	3,478,361.90	870,331.32	26.3		26.3						2,548,366.76	New Jersey
New Mexico	1,949.0	4,061,089.18	2,690,988.30	189.4	68.0	247.4	385,796.30	323,051.87	14.3	.2	14.6	2,351,742.67	New Mexico
New York	2,694.3	23,481,321.76	4,582,356.00	308.8		308.6	6,307,365.10	1,272,196.50	85.4		85.4	11,774,405.54	New York
North Carolina	1,881.6	3,618,874.15	1,732,800.08	128.4	35.1	163.6	538,307.37	264,659.06	49.7	4.2	63.9	4,442,881.70	North Carolina
North Dakota	4,502.8	2,129,399.54	1,068,766.80	265.8	167.4	433.2	1,174,218.95	686,815.85	181.9	120.1	302.0	2,478,338.14	North Dakota
Ohio	2,415.1	17,235,055.81	5,676,877.94	271.9	25.2	297.1	816,363.50	227,881.15	3.1	1.9	6.0	4,625,110.64	Ohio
Oklahoma	1,892.8	6,972,041.75	2,856,679.90	189.7	87.6	277.6	680,607.86	294,619.60	40.8	7.3	48.1	2,826,318.47	Oklahoma
Oregon	1,201.0	6,993,561.23	3,524,937.44	243.0	81.9	324.9	618,982.65	339,544.74	16.6		15.6	1,787,010.80	Oregon
Pennsylvania	2,610.3	10,333,868.77	3,158,673.30	88.3		88.3	738,014.19	324,026.66	6.0		6.0	6,367,086.61	Pennsylvania
Rhode Island	203.2	2,075,922.82	870,964.74	32.7		32.7						528,546.33	Rhode Island
South Carolina	1,848.4	6,190,361.17	2,410,005.89	112.0	122.6	234.8	647,266.54	320,761.87	.2	19.9	20.1	1,300,684.58	South Carolina
South Dakota	3,699.8	3,694,629.41	2,069,190.25	347.2	88.6	416.7	760,477.36	418,262.45	114.2	19.6	134.0	2,282,747.96	South Dakota
Tennessee	1,313.0	4,629,445.80	2,097,233.60	179.0	41.6	220.6	806,731.88	403,365.83	19.7		19.7	3,286,924.38	Tennessee
Texas	8,909.6	14,479,488.69	6,032,738.68	654.9	185.0	739.9	2,111,876.75	819,064.59	65.3	48.2	113.6	10,236,080.62	Texas
Utah	986.9	1,221,740.61	903,647.91	82.4	43.9	136.3	637,205.22	456,203.41	15.9	66.9	71.8	1,518,504.87	Utah
Vermont	290.1	885,501.67	214,396.44	14.7	1.4	16.1	20,775.32	10,387.88				813,889.05	Vermont
Virginia	1,688.1	4,601,089.95	2,153,980.85	190.0	1.9	191.9	869,876.21	380,383.98	20.6	14.2	34.8	2,256,550.02	Virginia
Washington	978.4	3,144,127.11	1,366,300.00	60.1	38.3	98.4	944,944.38	413,400.00	35.3		36.3	2,545,064.16	Washington
West Virginia	709.3	6,126,542.61	1,872,504.98	121.9	41.2	163.1	738,669.65	323,078.27	7.0		7.0	1,107,727.72	West Virginia
Wisconsin	2,394.9	6,417,090.04	2,691,250.00	154.6	38.4	192.9	212,787.83	100,000.00	8.8		8.8	2,591,034.41	Wisconsin
Wyoming	1,735.8	2,473,263.97	1,649,663.85	183.2	107.2	290.4	118,395.47	77,726.61	3.6	16.3	19.9	1,679,180.43	Wyoming
Hawaii	41.2	882,655.31	374,008.63	21.7		21.7	187,730.75	98,865.37	9.0		8.0	1,862,769.69	Hawaii
TOTALS	67,640.6	254,934,768.13	110,960,487.07	7,784.9	2,385.4	10,170.3	47,983,962.18	20,153,046.78	1,647.6	637.6	2,186.4	156,383,876.90	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 11 JANUARY, 1931

TEST SECTION ON TENNESSEE CONCRETE-CURING PROJECT

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

CERTIFICATE: By direction of the Secretary of Agriculture, the matter contained herein is published as administrative information and is required for the proper transaction of the public business

The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 11, NO. 11 JANUARY, 1931 G. P. St. CLAIR, Editor

TABLE OF CONTENTS

Page
A Study of Methods of Curing Concrete Pavements 209
Recent Publications of the American Society for Testing Materials 235

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A STUDY OF METHODS OF CURING CONCRETE PAVEMENTS

REPORT OF A FIELD INVESTIGATION IN TENNESSEE CONDUCTED JOINTLY BY THE TENNESSEE DEPARTMENT OF HIGHWAYS AND THE U. S. BUREAU OF PUBLIC ROADS

By F. H. Jackson Senior Engineer of Tests, United States Bureau of Public Roads, and E. W. Bauman, Engineer of Materials, Tennessee Department of Highways

THE field study of methods of curing concrete pavements reported herein was initiated by the Tennessee Department of Highways and Public Works for the purpose of determining the relative efficiency under service conditions of the various special curing methods which have been proposed from time to time as substitutes for the standard earth and water method. The department recognized the possibility of merit in these special methods, from the standpoint of effectiveness as well as that of cost. It possessed, however, no data as to the efficiency of any of them under Tennessee climatic conditions. A field investigation was therefore proposed to include all of the methods of cure to which the attention of the department had been called prior to July 1, 1928, as well as certain additional methods involving the application of burlap kept saturated for various periods up to 96 hour with no other curing.

The United States Bureau of Public Roads also being interested in the development of additional information relative to the various methods of curing concrete pavements, an agreement was entered into by the Tennessee department and the bureau for the cooperative conduct of the investigation reported herein.

It was decided to conduct the experiment on a concrete pavement approximately 15 miles in length, to be constructed on State Route No. 15, in Fayette County, Tenn., 20 miles east of Memphis. The location chosen seemed desirable because of the very uniform subgrade condition (a fine silty loam) along the entire length of the highway. Weather Bureau records likewise indicated the probability of hot, dry weather during most of the construction season. An announcement of the cooperative project, including a description of the plans for conducting it, was carried in the November, 1928, issue of PUBLIC ROADS.

A supplemental agreement was entered into between the State of Tennessee and the contractor, the J. B. McCrary Engineering Corporation, of Atlanta, Ga., providing for a change in the cross-section of the slab from the standard 8–6–8 double parabolic section to an 8–7–8 section with the dowel bars across the center joint eliminated. Expansion joint spacing was increased from 50 to 200 feet and the joint width increased from ¾ to 1½ inches. It was further specified that the center strip stakes should be pulled after finishing operations so that each half of the pavement might be free to move without hindrance from the adjacent section. The contractor was relieved of the duty of curing that portion of the pavement on the south side of the longitudinal joint but was required to cure the other side (the north side) in accordance with the standard specifications which were in force at that time and which provided for the application of burlap kept wet for 24 hours followed by a 2-inch layer of earth kept wet for 10 days. The burlap was dry at the time it was applied to the pavement and was made wet by subsequent sprinkling. The specifications further provided that the earth cover remain in place for at least 18 days.

CURING METHODS LISTED

The south side of the pavement was cured by the various special methods listed in Table 1. These were applied successively to sections approximately 1,000 feet in length, usually one day's run. This procedure permitted the repetition of most of the curing methods a number of times and therefore made it possible to observe their behavoir at various seasons of the year from July to December.

SOIL CONDITIONS ENCOUNTERED ON THIS PROJECT. NOTE EROSION CHARACTERISTICS

TABLE 1.—*List of curing methods employed*

Method of curing	Number of cycles [1]	Total length, feet
Wet burlap applied for 12 hours	2	2,053
Wet burlap applied for 24 hours	6	5,007
Wet burlap applied for 48 hours	5	4,359
Wet burlap applied for 72 hours	4	4,139
Wet burlap applied for 96 hours	4	3,252
No cure	9	7,524
Sisalkraft paper applied for 24 hours [2]	3	2,963
Sodium silicate:		
Earth subgrade	3	2,215
Tar paper on subgrade	2	1,897
Calcium chloride, surface application:		
Earth subgrade	8	5,265
Tar paper on subgrade	5	4,481
Hunt Process:		
Earth subgrade	4	3,668
Tar paper on subgrade	3	2,824
Barber asphalt emulsion (Curcrete):		
Earth subgrade	4	4,137
Tar paper on subgrade	3	2,760
Headley asphalt emulsion:		
Earth subgrade	4	3,483
Tar paper on subgrade	3	2,354
Tarvia K. P.:		
Earth subgrade	2	1,810
Tar paper on subgrade	1	816
Tarvia B:		
Earth subgrade	3	1,081
Tar paper on subgrade	1	719
Poor earth cure	4	3,318
Ponding	3	2,205
Calcium chloride admixture:		
Earth subgrade	3	1,053
Tar paper on subgrade	3	765
Total	-------	74,148

[1] In this column is given the number of test sections to which each type of curing was applied.

[2] On one section (cycle No. 2) the Sisalcraft paper was applied for 48 hours.

This program, it will be noted, provided a series of experimental sections on which it was possible to compare the concrete cured by the experimental method with a section across the longitudinal joint similar to it in every respect except that it was cured by the standard method.

Each special curing treatment was applied strictly in accordance with specifications recommended by the manufacturers or sponsors of the materials employed. In practically all cases representatives of the several companies interested were present at least once during the application of their particular material.

QUALITY OF CONCRETE CAREFULLY CONTROLLED

All materials used in the concrete pavement met the requirements of the State specifications. One brand of Portland cement was used throughout the job. Fine and coarse aggregates were obtained from a single source. The results of typical tests of cement, fine aggregates and coarse aggregates are given in Table 2.

TABLE 2.—*Tests of materials*

1. Portland cement:	
Fineness, per cent retained on No. 200 sieve	14. 0
Time of set—	
Initial	5 hours, 15 minutes.
Final	7 hours, 25 minutes.
Soundness	Satisfactory.
Tensile strength, pounds per square inch—	
At 7 days	305
At 28 days	400
Chemical analysis—	
Loss on ignition, per cent	0. 80
Insoluble residue, per cent	0. 53
Sulphuric anhydride, per cent	1. 45
Magnesia, per cent	3. 10
2. Fine aggregate:	
Specific gravity	2. 62
Color plate	Lignite.
Loss by washing, per cent	0. 1
Strength ratio, per cent—	
At 7 days	110
At 28 days	125
Sieve analysis—	
Sieve No.—	Per cent retained
4	2. 4
8	9. 6
14	27. 4
28	66. 1
48	96. 9
100	99. 6
Fineness modulus	3. 020
3. Coarse aggregate:	
Specific gravity	2. 58
Per cent of wear	8. 0
Soundness	Satisfactory.
Absorption, per cent	1. 5
Sieve analysis—	
Sieve	Per cent retained
1½-inch	6
1-inch	42
¾-inch	60
⅜-inch	91
No. 4	99
Fineness modulus	7. 56

The concrete was proportioned so as to give an expected average crushing strength of 3,500 pounds per square inch at 28 days and, as the result of tests, a water-cement ratio of 0.75 was established as the correct value for this particular combination of materials. Table 3 gives the results of 28-day tests on field cylinders cast in connection with the routine control of the job with no reference to the special curing methods studied. Each value reported in this table represents the average of tests on three cylinders.

TABLE 3.—*Crushing strength of concrete. Routine control tests on 6 by 12 inch cylinders*

Date laid	Crushing strength	Date laid	Crushing strength	Date laid	Crushing strength
	Pounds per square inch		*Pounds per square inch*		*Pounds per square inch*
July 6	3, 310	Aug. 22	4, 053	Oct. 13	3, 190
July 7	3, 104	Aug. 23	4, 371	Oct. 15	3, 685
July 9	3, 417	Aug. 24	4, 648	Oct. 16	2, 826
July 10	3, 886	Aug. 25	3, 997	Oct. 18	2, 575
July 11	3, 783	Aug. 27	3, 634	Oct. 19	4, 491
July 12	3, 879	Aug. 28	2, 969	Oct. 20	3, 514
July 13	3, 909	Aug. 29	3, 762	Oct. 22	3, 602
July 14	3, 867	Aug. 30	3, 971	Oct. 24	3, 256
July 16	4, 152	Sept. 4	3, 946	Oct. 25	4, 698
July 18	3, 005	Sept. 5	4, 598	Oct. 26	3, 816
July 19	3, 667	Sept. 6	3, 985	Oct. 29	3, 998
July 20	4, 151	Sept. 7	4, 022	Oct. 30	2, 841
July 23	3, 827	Sept. 10	3, 714	Oct. 31	2, 854
July 24	3, 675	Sept. 11	3, 714	Nov. 1	3, 023
July 25	3, 504	Sept. 12	3, 457	Nov. 3	3, 301
July 26	3, 271	Sept. 13	3, 298	Nov. 5	3, 609
July 27	3, 805	Sept. 14	4, 023	Nov. 6	2, 162
July 30	3, 815	Sept. 17	3, 705	Nov. 7	4, 153
July 31	4, 119	Sept. 18	3, 905	Nov. 9	4, 311
Aug. 1	3, 218	Sept. 19	4, 426	Nov. 10	3, 696
Aug. 2	3, 799	Sept. 21	4, 000	Nov. 12	3, 311
Aug. 3	3, 260	Sept. 22	4, 409	Nov. 13	3, 978
Aug. 4	3, 419	Sept. 24	4, 057	Nov. 14	3, 317
Aug. 6	3, 840	Sept. 25	4, 865	Nov. 15	2, 827
Aug. 7	3, 564	Sept. 26	4, 202	Nov. 16	3, 545
Aug. 8	3, 501	Sept. 27	4, 215	Nov. 17	2, 575
Aug. 10	3, 871	Sept. 28	4, 263	Nov. 20	2, 772
Aug. 11	3, 180	Sept. 29	4, 073	Nov. 21	3, 217
Aug. 13	3, 551	Oct. 1	3, 752	Nov. 22	2, 960
Aug. 14	3, 693	Oct. 3	4, 005	Nov. 23	3, 358
Aug. 15	4, 049	Oct. 5	3, 426	Nov. 24	3, 367
Aug. 16	3, 700	Oct. 6	3, 467	Nov. 27	3, 539
Aug. 17	3, 496	Oct. 10	4, 011	Dec. 1	3, 613
Aug. 18	3, 690	Oct. 11	3, 638	Dec. 2	4, 077
Aug. 20	3, 892	Oct. 12	4, 094	Dec. 3	3, 449

It will be observed that, although the proportions, including water, were kept reasonably constant on this job, considerable variation in crushing strength resulted, probably because of variations in curing conditions to which the field specimens were subjected. These variable results emphasize the difficulty of attempting to control uniformity of strength by means of tests on specimens cured under job conditions.

CONTROL TESTS CONDUCTED THROUGHOUT

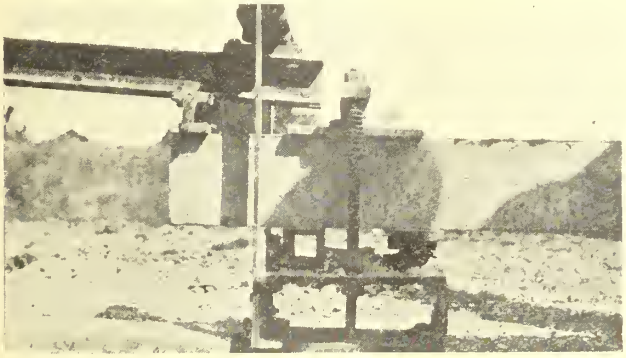
Control test specimens were constructed in the form of beams 6 by 6 by 42 inches in size, permitting four breaks with a cantilever-type transverse testing machine. Beams were cast in gangs of four, according to the procedure recommended in the report of Committee D–4, A. S. T. M., Proc. 1928, vol. 28, part 1, page 538. Twenty-eight beams were cast for each section, 12 cured as nearly as possible in the same manner as the standard earth and water cured section, 12 as nearly as possible in accordance with the special cure being used and 4 without any curing at all. The four so-called "no-cure" beams were tested in order to give some idea of the efficiency of both the standard earth and water cure and the special cures by comparison of cured specimens with specimens which had received no artificial curing at all. In all tests of beams the practice was followed of making one break from a given beam at each of four different ages; that is, 3, 7, 14, and 28 days. This practice provided an opportunity to study age-strength relations on individual specimens. The results of transverse tests of concrete, for each test condition and for a given age, represent, therefore, an average of one break from each of the 12 beams cast during that day's run.

The mass of transverse strength data resulting from these tests, some 10,000 individual beam breaks, was analyzed in the laboratory of the Tennessee Department of Highways by the method of least squares, and, as the result of this analysis, a value for modulus of rupture of concrete for each test condition was obtained which is designated the "most probable average" for the section. This appeared to be the most accurate method of eliminating wild or freak individual results.

When the curing method involved the spraying or painting of the material on the surface of the pavement, the method of curing the beams presented some difficulties. In this investigation it was the practice to spray or paint the surface of the beams in the specified manner. The forms were then stripped at the end of 24 hours, and the four beams constituting the set were placed on the subgrade with tar paper between the beams and earth banked around the outside edges. This method was criticised by certain of the manufacturers and as a result a special set of tests was run during the third cycle in which the sides of the beams were protected immediately after stripping the forms by means of an application of the curing material.

CONCRETE CORES FROM EACH SECTION TESTED

Four concrete cores were drilled from each test section throughout the project. Unusual difficulties encountered in drilling this concrete, due possibly to the presence of very hard flint in the coarse aggregate, prevented the testing of the cores at 90 days as originally contemplated. Cores were therefore drilled at various ages and shipped to the central laboratory at Nashville, where they were cured in moist air until the age of one year.



Beam in Testing Machine, Ready to be Broken

CRACK SURVEYS AND WEAR TESTS MADE

For the first two rounds of tests crack surveys were made on each section at frequent intervals during the first 60 days. In addition, two complete crack surveys were made at later periods, one on December 3, 1929, and one on May 26, 1930. These crack surveys show the average length of uncracked slab for each section for both the standard cure and the special cure.

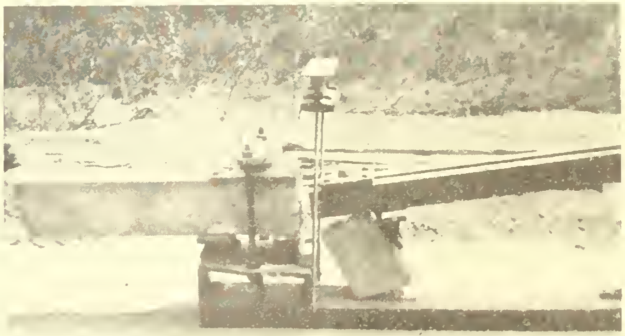
A series of surface wear tests on selected sections was also conducted in an effort to determine how the curing method affected this characteristic of the concrete. These tests were made with apparatus developed by the Bureau of Public Roads and first described in Public Roads, July, 1929.

DATA PRESENTED IN BOTH TABULAR AND GRAPHIC FORMS

The data obtained from the various tests are presented in Tables 4, 5, and 6 and Figures 1 and 2. In Table 4 are given the results of the strength tests of beams and cores. In the case of the beam tests values of the "most probable average" modulus of rupture at each age of test (3, 7, 14, and 28 days) are tabulated under the heading "Beam strength," for the specimens cured by experimental methods and the corresponding standard-cure and no-cure specimens. These values are given for each test section or cycle included in the investigation. Similarly, in the case of the core tests, values of core strength at one year are tabulated for both research and standard cured specimens.

In this and in subsequent tables values of the so-called test ratings are tabulated for both beam and core data. The test rating is defined as the comparative value or rating of a test result (strength, average slab length, etc.), expressed as the percentage which the value reported for the experimental cure bears to the corresponding value for the standard cure. In Table 4 an analogous rating is also given for comparison of the experimental with the "no-cure" beams. For purposes of reference the date of laying each test cycle, the length of cycle, the total rainfall during the 10-day period following laying of the concrete, and the average daily temperature range during that period are also included in Table 4. The temperatures given cover the working day rather than the whole 24 hours.

Moulded Beams, Ready for Application of Curing Material



First Break on Beam Specimen, Obtained 3 Days After Construction

TABLE 4.—*Results of strength tests on control test beams and on cores drilled from test sections*

Research cure	Cycle No.	Date laid	Location		Length of cycle	Precipitation during first 10 days after laying	Temperature first 10 days after laying		Beam strength in pounds per square inch (most probable average) [1]												Beam strength rating,[2] per cent								Core strength at one year		
			From station	To station			Average maximum	Average minimum	Research cure				Standard cure				No cure				Research cure to standard cure				Research cure to no cure				Average, research cure	Average, standard cure	Core strength rating [3]
									3-day	7-day	14-day	28-day	3-day	7-day	14-day	28-day	3-day	7-day	14-day	28-day	3-day	7-day	14-day	28-day	3-day	7-day	14-day	28-day			
					Feet	*Inches*	° *F.*	° *F.*																					*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Per cent.*
Burlap, 12 hours	1	July 5. 6, 7	267+58	281+50	1,392	3/8	93	76	408	452	541	482	424	462	535	513					96	98	101	89					3,352	3,066	109
	2	Nov. 20	658+54	665+15	661	2½	55	35	178	398	545	628	213	505	620	599	201	373	549	613	84	79	88	105	89	107	99	102	2,970	3,551	84
Average									293	425	543	550	319	484	578	571	201	373	549	613	90	88	95	97	89	107	99	102	3,161	3,308	96
Burlap, 24 hours	1	Aug. 24	113+38	123+60	1,022	2½	85	78	432	529	505	508	517	561	655	576	322	408	420	366	84	94	77	88	134	130	120	139	3,838	3,746	102
	2	July 9, 10	254+24	267+58	1,334	1½	94	74	394	470	530	530	429	489	517	572	282	407	389	492	92	96	102	93	140	115	136	108	3,787	3,094	122
	3	Oct. 29	564+90	567+75	285	1⅞	60	44	186	397	508	652	161	367	518	647	145	423	450	459	115	108	98	101	128	94	113	142	3.767	4,409	85
	4	Sept. 24	298+32	305+03	671	1½	81	59	392	466	488	570	427	560	582	692	224	265	243		92	83	84	82	175	176	201		3,880	3,880	100
	5	Nov. 1. 3	584+70	592+48	778	¾	60	41	168	415	516	618	149	410	514	629					113	101	100	98					4,022	3,563	113
	6	Nov. 24	623+38	632+55	917	2½	53	36	87	391	506	675	82	385	529	679	93	437	559	581	106	102	96	100	94	89	91	116	4,361	3,805	115
Average									277	445	509	592	294	462	553	632	213	388	412	475	100	97	93	94	134	121	132	126	3,942	3,750	106
Burlap, 48 hours	1	Aug. 25	123+60	133+65	1,005	2½	83	79	528	534	515	487	492	616	634	662	396	472	440	343	107	87	81	74	133	113	117	142	3,063	3,246	94
	2	July 11, 12	246+20	254+24	804	2	94	73	432	502	558	468	441	523	551	524	334	377	447	434	98	96	101	89	129	133	125	108	3,228	3,160	102
	3	Sept. 25	305+03	315+11	1,008	2¼	82	61	355	520	517	555	340	574	562	672	264	341	446		104	91	92	83	134	152	116		3,640	3,863	94
	4	Nov. 5	592+48	600+72	824	¾	62	45	283	482	574	629	303	552	609	693	257	425	563	426	93	87	94	91	110	113	102	148	3,796	3,762	101
	5	Nov. 27	613+54	620+72	718	2	61	46																					4,252	3,863	110
Average									400	510	541	535	394	566	589	638	313	404	474	401	100	90	92	84	127	128	115	133	3,596	3,579	100
Burlap, 72 hours	1	Aug. 27	133+65	144+36	1.071	2½	81	70	506	539	468	458	495	540	522	600	476	487	393	410	102	100	90	76	106	111	119	112	3,630	3,662	99
	2	July 13	235+80	246+20	1.040	1⅞	93	73	504	489	574	521	431	441	531	501	423	425	528	465	117	111	108	104	119	115	109	112	3,799	3,796	100
	3	Sept. 26	315+11	323+78	867	2¼	82	63	391	508	495	634	357	506	530	601	416	508	481		110	100	93	105	94	100	103		3,953	4,120	96
	4	Nov. 6, 7, 8	728+87	740+42	1.161	1½	65	48	159	428	544	611	176	416	542	559	125	368	440	588	90	103	100	109	127	116	124	104	4,067	2,712	150
Average									390	491	520	556	365	476	531	565	360	447	461	488	105	104	98	98	112	110	114	110	3,862	3,573	111
Burlap, 96 hours	1	Aug. 30	165+84	174+51	867	2½	81	61	490	576	515	548	471	567	617	617	403	386	397		104	102	84	89	122	149	130		3,777	4,122	92
	2	July 14, 16	227+92	235+80	788	2	92	74	437	531	644	532	399	516	573	572	412	646	547	488	110	103	112	93	106	82	118	109	3.305	3,176	104
	3	Sept. 29	343+57	349+64	607	2¼	83	65	341	438	427	513	306	436	477	534					111	100	90	96					3,255	3,902	83
	4	Nov. 10	712+48	722+38	990	1½	64	48	190	425	569	699	207	487	603	708	179	310	380	538	92	87	94	99	106	137	150	130	3,184	3,763	85
Average									365	493	539	573	346	502	568	608	331	447	441	513.	104	98	95	94	111	123	133	120	3,380	3,741	91
No cure	1	Aug. 15	55+55	62+50	695	2⅞	90	75	559	569	530	580	561	623	618	673	436	533	387	579	100	91	86	86	128	107	137	100	3,420	4,392	78
	2	Aug. 28	144+36	155+53	1,117	2½	81	66	398	429	485	460	444	507	510	499	394	406	502	476	90	85	95	92	101	106	97	97	3,433	3,167	108
	3	July 17, 18, 19	217+05	227+92	1,087	⅞	90	75	447	524	525	507	507	582	634	550					88	90	83	92					2,975	3,359	89
	4	Sept. 27	323+78	334+33	1,055	2¼	82	64	332	462	470	544	343	421	465	570	356	492	524	482	97	110	101	95	93	94	90	113	4,113	3,441	120
	5	Oct. 5, 6	369+21	381+80	1,259	2	86	67	405	451	538	552	444	579	675	718	432	333	450	527	91	78	80	77	94	135	120	105	3,090	3,717	83
	6	Sept. 13	425+00	428+22	322	0	82	62	404	430	426	498	481	595	671	637	426	460	410	518	81	72	64	78	95	93	104	96			
	7	Oct. 24	543+15	549+13	598	2⅞	64	47	361	571	557	640	365	616	649	736	379		571	652	99	93	86	87	95		98	98	4,825	3.986	121
	8	Nov. 23	632+55	639+97	742	2½	55	38	330	387	572	642	162	430	618	684	342	387	529	594	204	90	93	94	96	100	108	108	4,130	3,451	120
	9	Nov. 9	722+38	728+87	649	1½	64	48	204	402	515	588	180	464	623	670	296	405	470	604	113	87	83	88	69	99	110	97	3,624	3,978	91
Average									382	469	513	557	387	535	607	637	383	431	480	554	107	88	86	88	96	105	108	102	3,701	3,686	101
Sisalkraft, 24 hours	1	Aug. 29	155+53	165+84	1,031	2½	80	61	491	482	510	503	522	573	591	574	549	595	527	391	94	84	86	88	89	81	97	129	3,573	4,073	88
Sisalkraft, 48 hours	2	July 24	190+86	200+94	1,008	½	89	76	454	522	420	598	464	563	580	604	426		366	409	98	93	72	99	107		115	146	3,271	3,368	97
Sisalkraft, 24 hours	3	Sept. 28	334+33	343+57	924	2¼	83	65	449	538	492	564	460	517	501	626	381	476	425	612	98	104	98	90	118	113	116	92	4,551	4,131	110
Average									465	514	474	555	482	551	557	601	452	536	439	471	97	94	85	92	105	97	109	122	3,798	3,857	98

Sodium silicate, earth subgrade.	1	July 20	208+82	217+05	823	⅞	88	76	566	598	564	560	560	621	662	629	506	525	452	542	101	96	85	89	112	114	125	103	3,352	3,243	103
	2	Sept. 6	470+16	481+03	1,087	0	89	68	475	448	501	655	511	532	618	658	416	399	430	539	93	84	81	100	114	112	117	122	3,796	4,435	86
	3	Oct. 10	498+00	501+05	305	3½	80	62	408	521	550	553	400	493	581	583	470	488	604	493	102	106	95	95	87	107	91	112	2,612	4,902	53
Average									483	522	538	589	490	549	620	623	464	471	495	525	99	95	87	95	104	111	111	112	3,253	4,193	81
Sodium silicate, tar paper on subgrade.	1	July 23	200+94	208+82	788	½	89	76	422	520	557	538	436	565	598	593	487	568	599	567	97	92	93	91	87	92	93	95	3,006	3,102	97
	2	Sept. 5	481+03	492+12	1,109	0	88	67	484	426	489	656	448	563	602	654	444	424	464	480	108	76	81	100	109	100	105	137	3,263	4,379	75
Average									453	473	523	597	442	564	600	624	466	496	532	424	103	84	87	96	98	96	99	116	3,135	3,741	86
Calcium chloride, surface application, earth subgrade.	1	Aug. 16	62+50	65+80	330	2⅞	90	75	464	457	372	346	599	616	643	707	529	512	500	493	77	74	58	49	88	89	74	70	3,609	3,386	107
	2	July 25, 26	174+51	190+86	1,635	⅜	90	76	490	531	509	588	487	531	548	613	485	387	486	548	101	100	93	96	101	137	105	107	3,618	2,845	127
	3	Oct. 1	349+64	357+43	779	2¼	84	68	442	466	551	657	443	529	568	672	436	482	502	626	100	88	97	98	101	97	110	105	3,708	3,163	117
	4	Sept. 4	492+12	498+00	588	0	87	66	442	438	488	577	483	590	560	596	405	339	423	462	92	74	87	97	109	129	115	125	4,109	4,263	96
	5	Oct. 31	572+82	574+80	198	1⅞	59	44	131	353	419	534	89	356	401	510	209	407	467	----	147	99	104	105	63	87	90	----	2,481	3,889	64
	6	Nov. 21	649+63	658+54	891	2½	54	36	260	458	551	635	227	484	581	638	210	366	547	628	114	95	95	100	124	125	101	101	3,235	3,148	103
	7	Nov. 14	680+21	682+40	219	1½	62	45																					2,599	3,250	80
	8	Nov. 13	695+50	701+75	625	1½	63	45	391	565	504	582	385	572	611	604	363	482	424	468	102	99	82	96	108	117	119	124	3,380	2,835	119
Average									374	467	485	560	388	525	559	620	377	425	478	538	105	90	88	92	99	112	102	105	3,342	3,347	102
Calcium chloride, surface application, tar paper on subgrade.	1	July 30, 31	563+98	575+37	1,139	¼	93	79	458	533	522	530	500	613	653	688	518	609	555	579	92	87	80	77	88	88	94	92	4,032	4,371	92
	2	Oct. 3, 4	357+43	369+21	1,178	¾	86	66	432	593	608	666	476	590	605	698	434	454	534	644	91	100	100	96	100	131	114	103	3,891	4,124	94
	3	Oct. 31	577+07	582+00	493	1⅞	59	44	214	478	560	608	150	414	509	609	----	----	----	----	143	115	110	100	----	----	----	----	3,030	3,797	80
	4	Nov. 22	639+97	649+63	966	2½	54	35	207	423	571	634	189	488	574	636	138	284	729	640	109	87	100	100	150	149	78	99	3,139	3,937	80
	5	Nov. 14	684+40	691+45	705	1½	62	45	298	480	535	549	289	525	621	481	362	542	438	----	103	91	86	114	82	89	122	----	3,225	3,018	107
Average									322	501	559	597	321	526	592	622	363	472	564	621	108	96	94	96	105	114	102	98	3,463	3,849	91
Hunt process, earth subgrade.	1	Aug. 1, 2	575+37	589+96	1,459	0	92	78	433	476	557	481	445	566	586	620	392	515	429	449	97	84	95	78	110	92	130	107	3,444	4,692	73
	2	Sept. 7	462+16	470+16	800	0	89	69	464	542	492	558	484	551	593	621	480	468	493	530	96	98	83	90	97	116	100	105	3,916	4,564	86
	3	Oct. 19	534+97	538+33	336	3¼	70	50	483	585	612	593	415	617	625	650	374	513	625	544	116	95	98	91	129	114	98	109	4,163	4,746	88
	4	Nov. 12	701+75	712+48	1,073	1½	63	46	380	547	633	646	323	565	699	717	205	347	410	505	118	97	91	90	185	158	154	128	3,377	3,602	94
Average									440	538	574	570	417	575	626	652	363	461	489	507	107	94	92	87	130	120	121	112	3,725	4,401	85
Hunt process, tar paper on subgrade.	1	Aug. 3, 4	589+96	9+05	1,367	0	92	77	418	516	517	490	487	553	582	371	439	439	441	419	86	93	89	132	95	118	117	117	3,504	4,093	86
	2	Sept. 10	452+41	462+16	975	0	86	65	493	498	500	501	468	553	605	504	410	425	378	361	105	90	83	99	120	117	132	139	3,711	3,757	99
	3	Oct. 20	538+33	543+15	482	3¼	68	49	422	622	668	652	393	605	650	656	296	432	474	507	107	103	103	99	143	144	141	129	3,858	4,560	86
Average									444	545	562	548	449	570	612	510	382	432	431	429	99	95	92	110	119	126	130	128	3,691	4,117	90
Barber asphalt emulsion, earth subgrade.	1	Aug. 18	71+84	82+00	1,016	2⅛	89	76	451	449	467	506	391	445	520	534	475	410	439	466	115	101	90	95	95	110	106	109	3,508	3,566	98
	2	Sept. 11	441+44	452+41	1,097	0	86	64	478	478	491	576	520	578	602	585	385	451	249	374	92	83	82	98	124	106	197	154	4,177	3,501	119
	3	Oct. 11	501+05	511+08	1,003	3½	79	61	495 463	538 543	636 650	592 597	461 461	544 544	670 670	647 647	503 503	465 465	491 491	502 502	107 100	99 100	95 97	92 92	98 92	116 117	130 132	118 119	3,770	4,361	86
	4	Nov. 15, 16	670+00	680+21	1,021	1½	56	38	329	325	577	575	356	558	596	514	436	475	586	575	92	58	97	112	75	68	98	100	3,595	3,485	103
Average									438	448	543	562	432	531	597	570	450	450	441	479	101	85	91	99	98	100	133	120	3,762	3,728	101
Barber asphalt emulsion, tar paper on subgrade.	1	Aug. 20	82+00	92+03	1,003	2⅛	89	76	508	451	481	491	474	520	547	592	448	382	541	----	107	87	88	83	113	118	89	----	3,138	3,550	88
	2	Sept. 12	430+81	441+44	1,063	0	84	63	470	503	518	556	483	543	586	580	466	386	382	407	97	93	88	96	101	130	136	137	3,640	4,454	82
	3	Oct. 12	511+08	518+02	694	5	78	60	454 430	531 574	583 656	614 615	428 428	608 608	662 662	667 667	403 403	504 504	591 591	576 576	106 100	87 94	88 99	92 92	113 107	105 114	99 111	107 107	3,645	4,135	88
Average									477	495	527	554	462	557	598	613	439	424	505	492	103	89	88	90	109	118	108	122	3,474	4,046	86
Headley asphalt emulsion, earth subgrade.	1	Aug. 21, 22	92+03	102+88	1,085	2⅝	87	78	400	497	529	514	557	533	588	627	397	414	540	404	72	93	90	82	101	120	98	127	3,140	3,459	91
	2	Sept. 21	281+50	290+59	909	0	79	52	495	558	542	565	510	611	582	696	265	327	406	431	97	91	93	81	187	171	134	131	3,346	3,395	99
	3	Oct. 25	549+13	559+17	1,004	2⅞	62	46	397 344	541 592	620 530	671 588	392 392	630 630	694 694	750 750	286 286	437 437	535 535	476 476	101 88	86 94	89 76	89 78	139 120	124 135	116 99	141 124	4,213	4,328	97
	4	Nov. 17	665+15	670+00	485	1½	53	35	264	458	518	555	237	421	594	573	----	----	----	----	111	109	87	97	----	----	----	----	3,348	3,534	95
Average									389	514	552	576	424	549	615	662	316	393	493	437	95	95	90	87	142	138	116	133	3,512	3,679	95
Headley asphalt emulsion, tar paper on subgrade.	1	Aug. 23	102+88	113+38	1,050	2⅝	87	78	466	504	517	573	496	517	604	611	447	431	525	674	94	98	86	94	104	117	98	85	3,248	4,001	81
	2	Sept. 22	290+59	298+32	773	½	79	54	478	557	549	643	412	502	619	575	506	585	566	700	116	111	89	112	94	95	97	92	3,416	3,543	96
	3	Oct. 26	559+17	564+48	531	2⅞	61	45	297 351	572 530	676 683	745 782	251 251	567 567	717 717	753 753	293 293	564 564	642 642	781 781	118 140	101 94	94 95	99 104	101 120	101 94	105 106	95 100	4,025	4,673	86
Average									414	544	581	654	386	529	647	646	415	527	578	718	109	103	90	102	100	104	100	91	3,563	4,772	88
Tarvia K. P., earth subgrade.	1	Aug. 11, 13	31+32.5	46+84	1,551.5	¾	90	74	482	536	531	523	435	521	513	562	476	579	557	461	111	103	104	93	101	93	95	113	3,993	4,515	88
	2	Sept. 13	428+22	430+81	259	0	82	62	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	----	3,334	4,638	72
Average									482	536	531	523	435	521	513	562	476	579	557	461	111	103	104	93	101	93	95	113	3,664	4,577	80

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[1] Where two sets of results are shown for a given section the results shown on the upper line represent tests of beams coated on top only, while those on the lower line represent tests of beams coated on sides as well as on top. The latter tests (3 sides coated) are not included in the average.

[2] Beam and core strength ratings are expressed as the percentages which the strengths of the research-cured specimens bear to the strengths of the corresponding standard-cured sections. Average strength ratings for a given type of cure were obtained by averaging the strength ratings for the individual cycles.

TABLE 4.—*Results of strength tests on control test beams and on cores drilled from test sections*—Continued

Research cure	Cycle No.	Date laid	Location		Length of cycle	Precipitation during first 10 days after laying	Temperature first 10 days after laying		Beam strength in pounds per square inch (most probable average)												Beam Strength rating, per cent								Core strength at one year		
			From station	To station			Average maximum	Average minimum	Research cure				Standard cure				No cure				Research cure to standard cure				Research cure to no cure				Average, research cure	Average, standard cure	Core strength rating
									3-day	7-day	14-day	28-day	3-day	7-day	14-day	28-day	3-day	7-day	14-day	28-day	3-day	7-day	14-day	28-day	3-day	7-day	14-day	28-day			
					Feet	*Inches*	° *F.*	° *F.*																					*Lbs. per sq. in.*	*Lbs. per sq. in.*	*Per cent.*
Tarvia K. P., tar paper on subgrade.	1	Aug. 14	46+84	55+00	816	3⅞	90	74	489	541	573	427	444	513	586	574	----	----	----	----	110	106	98	74	----	----	----	----	3,116	3,952	79
Average									489	541	573	427	444	513	586	574	----	----	----	----	110	106	98	74	----	----	----	----	3,116	3,952	79
Tarvia B, earth subgrade	1	Sept. 17	413+50	417+90	440	0	77	52	391	429	467	490	413	535	584	544	234	297	297	355	95	80	80	90	167	144	157	138	4,106	4,278	96
	2	Sept. 14	417+90	421+61	371	0	80	57	384	438	492	521	408	559	574	523	----	----	----	----	94	78	86	100	----	----	----	----	4,131	3,848	107
	3	Nov. 1	582+00	584+70	270	1⅝	58	44	249	428	469	686	301	474	529	649	328	463	448	600	83	90	89	106	76	92	105	114	3,490	3,678	95
									283	475	458	634	301	474	529	649	328	463	448	600	94	100	87	98	86	103	102	106	----	----	----
Average									341	432	476	566	374	523	562	572	281	380	373	478	91	83	85	99	122	118	131	126	3,909	3,935	99
Tarvia B, tar paper on subgrade.	1	Sept. 17	406+31	413+50	719	0	77	52	457	545	527	525	412	548	629	600	289	309	216	418	111	99	84	88	158	176	244	126	3,480	4,610	75
Average									457	545	527	525	412	548	629	600	289	309	216	418	111	99	84	88	158	176	244	126	3,480	4,610	75
Poor earth cure	1	Aug. 6, 7, 8	9+05	21+85	1,280	1¾	92	75	499	534	577	546	551	587	592	649	401	325	489	486	91	91	97	84	124	164	118	112	3,890	4,678	83
	2	Sept. 18	395+10	406+31	1,121	0	78	52	393	432	565	575	370	500	590	605	442	397	540	605	106	86	96	95	89	109	105	95	3,918	3,880	101
	3	Oct. 13	518+02	521+48	346	5	76	59	363	539	609	608	331	491	579	552	----	----	----	----	110	110	105	110	----	----	----	----	4,420	4,128	107
	4	Oct. 18	529+26	534+97	571	2½	71	50	459	588	649	631	445	536	648	608	401	456	535	513	103	110	100	104	114	129	121	123	4,067	3,870	105
Average									429	523	600	590	424	529	602	604	415	393	521	535	102	99	100	98	109	134	115	110	4,074	4,139	99
Ponding	1	Aug. 9, 10	23+95	29+65	570	1¾	91	74	414	466	467	486	420	492	516	548	429	393	486	498	99	95	90	89	97	119	96	98	4,020	4,147	97
	2	Sept. 19	381+80	395+10	1,330	0	79	53	457	517	617	602	486	622	643	629	423	416	492	505	94	83	96	96	108	124	125	119	4,267	3,940	108
	3	Oct. 15, 16	521+48	524+53	305	3	73	52	433	579	611	651	365	578	614	664	472	626	588	656	119	100	100	98	92	92	104	99	3,859	3,928	98
Average									435	521	565	580	424	564	591	614	441	478	522	553	104	93	95	94	99	112	108	105	4,049	4,005	101
Calcium chloride admixture, earth subgrade.	1	Aug. 16	65+80	68+46	266	2⅞	90	75	491	470	378	399	599	616	643	707	529	512	500	493	82	76	59	56	93	92	76	81	3,372	3,693	91
	2	Oct. 30	567+75	572+82	507	1⅞	60	44	309	381	467	512	161	367	518	647	145	423	450	459	192	104	90	79	213	90	104	112	3,454	3,207	108
	3	Nov. 13	692+70	695+50	280	1½	62	45	373	533	471	548	385	572	611	604	363	482	424	468	97	93	77	91	103	111	111	117	3,745	2,714	138
Average									391	461	439	486	382	518	591	653	346	472	458	473	124	91	75	75	136	98	97	103	3,524	3,205	112
Calcium chloride admixture, tar paper on subgrade.	1	Aug. 17	68+46	71+84	338	2⅛	89	76	441	438	402	403	391	445	520	534	475	410	439	466	113	98	77	75	93	107	92	86	3,030	3,169	96
	2	Oct. 31	574+80	577+07	227	1⅞	59	44	304	461	558	543	89	356	401	510	209	407	467	----	342	129	139	106	145	113	119	----	3,438	4,163	83
	3	Nov. 14	682+40	684+40	200	1½	62	45	471	550	590	----	289	525	621	481	362	542	438	----	163	105	95	----	130	101	135	----	3,218	3,427	94
Average									405	483	517	473	256	442	514	508	349	453	448	466	206	111	104	90	123	107	115	86	3,229	3,586	91

TABLE 5.—*Results of crack surveys*

Type of research cure	Cycle No.	Date laid	Length of section	Survey of Dec. 3, 1929			Survey of May 26, 1930		
				Average length of section		Slab length rating [1]	Average length of section		Slab length rating [1]
				Research cure	Standard cure		Research cure	Standard cure	
			Feet	*Feet*	*Feet*	*Per cent*	*Feet*	*Feet*	*Per cent*
Burlap, 12 hours	1	July 5, 6, 7	1,392	53.6	55.7	96.2	51.6	53.6	96.3
	2	Nov. 20	661	50.8	60.1	84.6	44.1	55.1	80.0
Average				52.7	57.0	92.3	48.9	54.0	90.5
Burlap, 24 hours	1	Aug. 24	1,022	63.9	53.8	118.8	63.9	53.8	118.8
	2	July 9, 10	1,334	58.0	60.6	95.6	58.0	58.0	100.0
	3	Oct. 29	285	71.2	71.2	100.0	71.3	71.3	100.0
	4	Sept. 24	671	95.9	95.9	100.0	83.9	83.9	100.0
	5	Nov. 1, 3	778	77.8	77.8	100.0	70.7	77.8	90.9
	6	Nov. 24	917	45.8	61.2	75.0	43.7	50.9	85.7
Average				62.6	65.0	96.2	60.4	61.1	98.8
Burlap, 48 hours	1	Aug. 25	1,005	47.9	41.9	114.3	47.9	40.2	119.0
	2	July 11, 12	804	57.4	50.2	114.3	50.2	47.3	106.2
	3	Sept. 25	1,008	77.5	84.0	92.3	72.0	77.6	92.8
	4	Nov. 5	824	82.4	82.4	100.0	82.4	74.9	110.0
	5	Nov. 27	718	35.9	37.8	95.0	32.6	32.6	100.0
Average				55.9	53.8	103.9	52.5	49.5	106.0
Burlap, 72 hours	1	Aug. 27	1,071	53.6	44.6	120.0	46.6	41.2	113.0
	2	July 13	1,040	69.3	57.8	120.0	65.0	57.8	112.5
	3	Sept. 26	867	61.9	57.8	107.1	57.8	54.2	106.6
	4	Nov. 6, 7, 8	1,161	68.3	64.5	105.9	68.3	64.5	105.9
Average				62.7	55.2	113.7	58.3	53.1	109.8
Burlap, 96 hours	1	Aug. 30	867	54.2	57.7	93.7	54.2	54.2	100.0
	2	July 14, 16	788	52.5	60.6	86.7	52.5	60.6	86.7
	3	Sept. 29	607	60.7	55.2	110.0	60.7	46.7	130.0
	4	Nov. 10	990	52.1	58.2	89.4	52.1	55.0	94.8
Average				54.2	58.1	93.4	54.2	54.2	100.0
No cure	1	Aug. 15	695	49.6	53.4	92.8	43.4	53.4	81.3
	2	Aug. 28	1,117	48.6	48.6	100.0	48.6	48.6	100.0
	3	July 17, 18, 19	1,087	64.0	49.4	129.4	63.9	49.4	129.4
	4	Sept. 27	1,055	48.0	50.2	95.4	48.0	50.2	95.4
	5	Oct. 5, 6	1,259	74.0	74.0	100.0	74.0	70.0	105.8
	6	Sept. 13	322	46.0	29.3	157.1	46.0	29.3	157.1
	7	Oct. 24	598	85.4	74.8	114.3	85.4	74.8	114.3
	8	Nov. 23	742	30.9	43.7	70.8	28.5	41.2	69.2
	9	Nov. 9	649	81.1	81.1	100.0	64.9	81.1	80.0
Average				54.1	53.8	100.7	51.9	53.0	97.9
Sisalkraft, 24 hours	1	Aug. 29	1,031	41.3	44.8	92.0	36.8	43.0	85.7
Sisalkraft, 48 hours	2	July 24	1,008	56.0	56.0	100.0	56.0	56.0	100.0
Sisalkraft, 24 hours	3	Sept. 28, 29	924	57.8	61.6	93.7	54.4	54.4	100.0
Average				50.2	52.9	94.9	47.0	50.2	93.7
Sodium silicate, earth subgrade	1	July 20	823	63.3	54.9	115.4	54.9	54.9	100.0
	2	Sept. 6	1,087	40.3	47.3	85.2	40.2	47.3	85.2
	3	Oct. 10	305	43.6	43.6	100.0	38.1	33.9	112.5
Average				47.1	49.2	95.8	44.3	47.1	94.0
Sodium silicate, tar paper on subgrade	1	July 23	788	49.2	56.3	87.2	43.8	49.2	88.9
	2	Sept. 5	1,109	44.4	55.4	80.0	42.6	48.2	88.5
Average				46.3	55.8	82.9	43.1	48.7	88.7
Calcium chloride surface application, earth subgrade	1	Aug. 16	330	30.0	33.0	90.9	27.5	30.0	91.6
	2	July 25, 26	1,635	56.4	58.4	96.6	51.1	54.5	93.7
	3	Oct. 1	779	86.6	97.4	88.9	86.5	97.4	88.9
	4	Sept. 4	588	42.0	45.2	92.8	39.2	39.2	100.0
	5	Oct. 31	198	66.0	66.0	100.0	66.0	66.0	100.0
	6	Nov. 21	891	40.5	46.9	86.4	37.1	42.4	87.5
	7	Nov. 14	219	73.0	73.0	100.0	54.8	73.0	75.0
	8	Nov. 13	625	78.1	78.1	100.0	69.4	78.1	88.9
Average				53.2	57.2	93.0	48.8	53.3	91.6
Calcium chloride surface application, tar paper on subgrade	1	July 30, 31	1,139	34.5	38.0	90.9	31.6	36.7	86.2
	2	Oct. 3-4	1,178	69.3	65.4	105.9	69.3	65.4	105.9
	3	Oct. 31	493	70.4	70.4	100.0	54.8	44.8	122.2
	4	Nov. 22	966	48.3	46.0	105.0	38.6	43.9	88.0
	5	Nov. 14	705	88.1	88.1	100.0	88.2	88.2	100.0
Average				52.7	53.4	98.8	47.2	49.8	94.8
Hunt process, earth subgrade	1	Aug. 1, 2	1,459	31.0	34.7	89.4	28.6	33.2	86.4
	2	Sept. 7	800	26.7	38.1	70.0	23.5	38.1	61.8
	3	Oct. 19	336	22.4	37.3	60.0	22.4	37.3	60.0
	4	Nov. 12	1,073	21.5	46.7	46.0	19.5	43.0	45.5
Average				25.8	38.6	66.9	23.7	37.1	63.8

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[1] The slab length rating is expressed as the percentage which the average length of research-cured section bears to the corresponding averago length of standard-cured section.

Table 5.—*Results of crack surveys*—Continued

Type of research cure	Cycle No.	Date laid	Length of section	Survey of Dec. 3, 1929			Survey of May 26, 1930		
				Average length of section		Slab length rating	Average length of section		Slab length rating
				Research cure	Standard cure		Research cure	Standard cure	
			Feet	*Feet*	*Feet*	*Per cent*	*Feet*	*Feet*	*Per cent*
Hunt process, tar paper on subgrade	1	Aug. 3, 4	1,367	27.3	38.0	72.0	24.4	37.0	66.1
	2	Sept. 10	975	29.5	33.6	87.9	25.6	30.4	84.2
	3	Oct. 20	482	48.2	40.2	120.0	48.2	40.2	120.0
Average				30.4	36.7	82.8	27.2	34.9	77.8
Barber asphalt emulsion, earth subgrade	1	Aug. 18	1,016	21.6	37.6	57.4	19.6	36.3	53.8
	2	Sept. 11	1,097	21.5	30.5	70.6	20.3	28.9	70.4
	3	Oct. 11	1,003	35.8	41.8	85.7	33.4	41.8	80.0
	4	Nov. 15, 16	1,021	46.4	51.0	90.9	44.4	51.1	87.0
Average				28.0	38.7	72.2	26.0	37.6	69.2
Barber asphalt emulsion, tar paper on subgrade	1	Aug. 20	1,003	25.1	40.1	62.5	22.3	37.2	60.0
	2	Sept. 12	1,063	25.9	36.7	70.7	23.6	33.2	71.2
	3	Oct. 12	694	36.5	49.6	73.6	34.7	49.6	70.0
Average				27.6	40.6	68.0	25.1	37.8	66.4
Headley asphalt emulsion, earth subgrade	1	Aug. 21, 22	1,085	20.9	31.0	67.3	20.1	31.0	64.8
	2	Sept. 21	909	35.0	47.8	73.1	34.9	37.8	92.4
	3	Oct. 25	1,004	24.5	47.8	51.2	22.8	40.2	56.8
	4	Nov. 16, 17	485	25.5	48.5	52.6	24.2	48.5	50.0
Average				25.2	41.0	61.6	24.2	37.1	65.2
Headley asphalt emulsion, tar paper on subgrade	1	Aug. 23	1,050	22.8	35.0	65.2	20.6	32.8	62.8
	2	Sept. 22	773	38.6	51.5	75.0	36.8	48.3	76.2
	3	Oct. 26	531	53.1	59.0	90.0	37.9	59.0	64.2
Average				31.0	43.6	71.1	27.4	41.3	66.3
Tarvia K. P., earth subgrade	1	Aug. 11, 13	1,551.5	26.3	40.8	64.4	24.2	39.8	60.9
	2	Sept. 13	259	16.2	23.5	68.7	14.4	19.9	72.2
Average				24.1	37.0	65.4	22.1	34.8	63.5
Tarvia K. P., tar paper on subgrade	1	Aug. 14	816	18.5	40.8	45.4	17.0	37.1	45.8
Average				18.5	40.8	45.4	17.0	37.1	45.8
Tarvia B, earth subgrade	1	Sept. 17	440	22.0	33.8	65.0	21.0	33.8	61.9
	2	Sept. 14	371	17.7	26.5	66.7	15.5	24.7	62.5
	3	Nov. 1	270	67.5	67.5	100.0	67.5	54.0	125.0
Average				24.0	34.9	68.9	22.1	32.8	67.4
Tarvia B, tar paper on subgrade	1	Sept. 17	719	37.8	47.9	78.9	36.0	45.0	80.0
Average				37.8	47.9	78.9	36.0	45.0	80.0
Poor earth cure	1	Aug. 6, 7, 8	1,280	55.6	45.7	121.8	47.4	41.3	114.7
	2	Sept. 18	1,121	70.1	74.8	93.8	70.1	74.8	93.8
	3	Oct. 13	346	86.5	86.5	100.0	86.5	86.5	100.0
	4	Oct. 18	571	81.6	95.2	85.7	63.5	95.2	66.7
Average				66.4	62.6	106.0	59.3	59.3	100.0
Ponding	1	Aug. 9, 10	570	40.7	43.8	92.8	38.0	43.8	86.7
	2	Sept. 19	1,330	63.4	78.2	80.9	55.4	66.5	83.3
	3	Oct. 15, 16	305	61.0	61.0	100.0	61.0	61.0	100.0
Average				55.1	63.0	87.5	50.1	58.0	86.4
Calcium chloride admixture, earth subgrade	1	Aug. 16	266	29.6	38.0	77.8	29.6	38.0	77.8
	2	Oct. 30	507	63.4	50.7	125.0	63.4	50.7	125.0
	3	Nov. 13	280	35.0	46.7	75.0	31.1	46.7	66.7
Average				42.1	45.8	92.0	40.5	45.8	88.5
Calcium chloride admixture, tar paper on subgrade	1	Aug. 17	338	37.6	42.2	88.9	37.6	42.2	88.9
	2	Oct. 31	227	56.7	56.8	100.0	45.4	56.8	80.0
	3	Nov. 14	200	100.0	100.0	100.0	100.0	100.0	100.0
Average				51.0	54.6	93.3	47.8	54.6	87.5

In Table 5 are presented the results of the crack surveys of December 3, 1929, and May 26, 1930. This tabulation includes values of average length of uncracked section for both research and standard cures, and the corresponding test ratings. The results of the tests for surface hardness are tabulated in a similar manner in Table 6, and are discussed in a subsequent part of the report.

The data presented in Tables 4 and 5 is shown in condensed graphic form in Figures 1 and 2. These two diagrams give the date of laying each test section, the daily rainfall and temperature records, and the various test ratings, with the exception of those for surface hardness given in Table 6. The temperature ranges, which cover the working day only, are denoted by vertical lines, the presence of an arrow indicating that no minimum temperature was recorded. The ratings for each section or cycle are also denoted by vertical lines, which appear in the following order, reading from left to right:

1. Modulus of rupture at 3 days.
2. Modulus of rupture at 7 days.
3. Modulus of rupture at 14 days.
4. Modulus of rupture at 28 days.
5. Crushing strength at 1 year.
6. Uncracked slab length on December 3, 1929.
7. Uncracked slab length on May 26, 1930.

SIGNIFICANCE OF TEST DATA DISCUSSED

Before the results are considered in detail, it will be of interest to discuss briefly the significance of the methods of establishing the relative efficiency of the various cures. The beam tests were made for the purpose of determining the effect of the curing method upon early strength; that is, upon strength acquired during periods up to 28 days. The value of a rating based on the strength of molded beams depends, of course, upon the degree to which it can be assumed that the curing effect on the beam is exactly the same as the effect upon the road slab itself. Although every effort was made in this work to make the conditions of curing closely comparable, it must be admitted that certain of the curing methods, especially those which depend upon sealing the moisture within the concrete, probably had an effect upon the small beams somewhat different from their effect upon the pavement because of the greater relative surface area exposed to evaporation. As has been noted above, the practice of spraying only the tops of the beams was criticized by the sponsors of certain of the so-called "black-surface" or bituminous cures. As a result, a number of special sets of beams were cast, which were sprayed on both sides and top, the sides being sprayed immediately after removal of the forms. The results of transverse tests on these beams compared to similar beams sprayed only on the top are given in Table 7. It will be observed that the additional spraying produced no consistent effect in either direction.

TABLE 6.—*Results of tests for surface hardness*

Type of cure	Cycle No.	Depth of wear, thousandths of an inch: Research cure	Depth of wear, thousandths of an inch: Standard cure	Wear test rating,[1] per cent
Wet burlap applied for 12 hours	1	35.3	39.3	90
Wet burlap applied for 24 hours	1	43.7	57.0	77
Wet burlap applied for 48 hours	1	34.3	42.3	81
Wet burlap applied for 72 hours	1	35.7	47.3	75
Wet burlap applied for 96 hours	1	43.7	58.7	74
No cure	1	33.3	37.7	88
Do	4	32.0	37.3	86
Sisalkraft paper applied for 24 hours	1	43.7	54.0	81
Sisalkraft paper applied for 48 hours	2	36.0	36.0	100
Sodium silicate, earth subgrade	1	25.3	27.3	93
Sodium silicate, tar paper on subgrade	2	32.7	58.7	56
Calcium chloride, surface application, earth subgrade	3	32.3	31.3	103
Calcium chloride, surface application, tar paper on subgrade	1	43.3	47.0	92
Hunt process, earth subgrade	1	29.0	30.3	96
Do	2	41.0	37.7	109
Do	4	45.2	28.0	161
Hunt process, tar paper on subgrade	1	53.7	34.0	158
Barber asphalt emulsion, earth subgrade	1	46.3	32.7	142
Do	2	49.7	49.7	100
Do	3	52.3	36.3	144
Barber asphalt emulsion, tar paper on subgrade	1	68.7	52.7	130
Headley asphalt emulsion, earth subgrade	3	70.0	34.3	204
Headley asphalt emulsion, tar paper on subgrade	2	53.0	28.7	185
Tarvia K. P., earth subgrade	1	38.0	26.0	146
Tarvia K. P., tar paper on subgrade	1	66.0	46.0	144
Tarvia B—earth subgrade	1	50.0	58.0	86
Do	2	77.0	56.5	136
Do	3	41.3	39.7	104
Poor earth cure	1	47.7	31.3	152
Ponding	1	45.7	44.3	103
Do	2	41.0	37.7	109
Calcium chloride admixture, earth subgrade	1	34.0	38.3	89
Do	2	52.3	50.7	103

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[1] The wear test rating is defined as the percentage which the depth of wear recorded for the research-cured side of the section bears to that recorded for the standard-cured side.

TABLE 7.—*Strength of beams cured on sides and top compared with beams cured on top only*

Type of cure	Strength of beams cured on top and sides, pounds per square inch				Strength of beams cured on top only, pounds per square inch				Strength of beams cured on top and sides as a percentage of strength of beams cured on top only			
	Age—Days				Age—Days				Age—Days			
	3	7	14	28	3	7	14	28	3	7	14	28
Barber asphalt emulsion, earth subgrade	463	543	650	597	495	538	636	592	94	101	102	101
Barber asphalt emulsion, tar paper on subgrade	430	574	656	615	454	531	583	614	95	108	113	100
Headley asphalt emulsion, earth subgrade	344	592	530	588	397	541	620	671	87	109	86	88
Headley asphalt emulsion, tar paper on subgrade	351	530	683	782	297	572	676	745	118	93	101	105
Tarvia B, earth subgrade	283	475	458	634	249	428	469	686	114	111	98	92

It is not believed that very much weight should be attached to the results of the beam tests because of the manifest impossibility of producing a curing condition strictly comparable with that of the pavement. It seems advisable that future experiments of this character should be conducted in the laboratory under standard conditions; or else that tests for strength should be made on specimens taken directly from the full size sections.

CORE TESTS CONSIDERED OF MORE IMPORTANCE THAN BEAM TESTS

The significance of the core strength data lies in the information which they afford relative to the influence of the curing methods on the ultimate strength of the concrete. Core tests obviously have one advantage over tests on molded specimens in that they are made on specimens cut from the actual structure. A disadvantage (common to all core tests on concrete pavements) lies in the fact that each individual core represents such a small portion of the slab, which at best is far from uniform in quality, that it is necessary to average a large number of individual tests before the results become truly significant. In this investigation it was possible to drill only four cores from each test section so that each result given in Table 4 represents the average of only four tests. The lack of a sufficient number of individual values from which to compute average results probably accounts for certain erratic and apparently inconsistent results appearing in the core test data. In spite of this, the authors are inclined to give the core data somewhat more weight than the beam test data because the specimens were actually taken from the full size structure and to this extent should represent the effect of the curing agent under actual conditions of use.

GREATEST WEIGHT GIVEN DATA REGARDING FORMATION OF CRACKS

In considering the data presented in this paper, the authors believe that the greatest weight should be given to the amount of cracking which has developed on the several test sections. The primary function of any curing method is to retain moisture in or supply moisture to the concrete from the time the slab is placed until sufficient strength has been built up to resist the stresses due to contraction when the slab finally is allowed to dry. Failures of the curing agent to

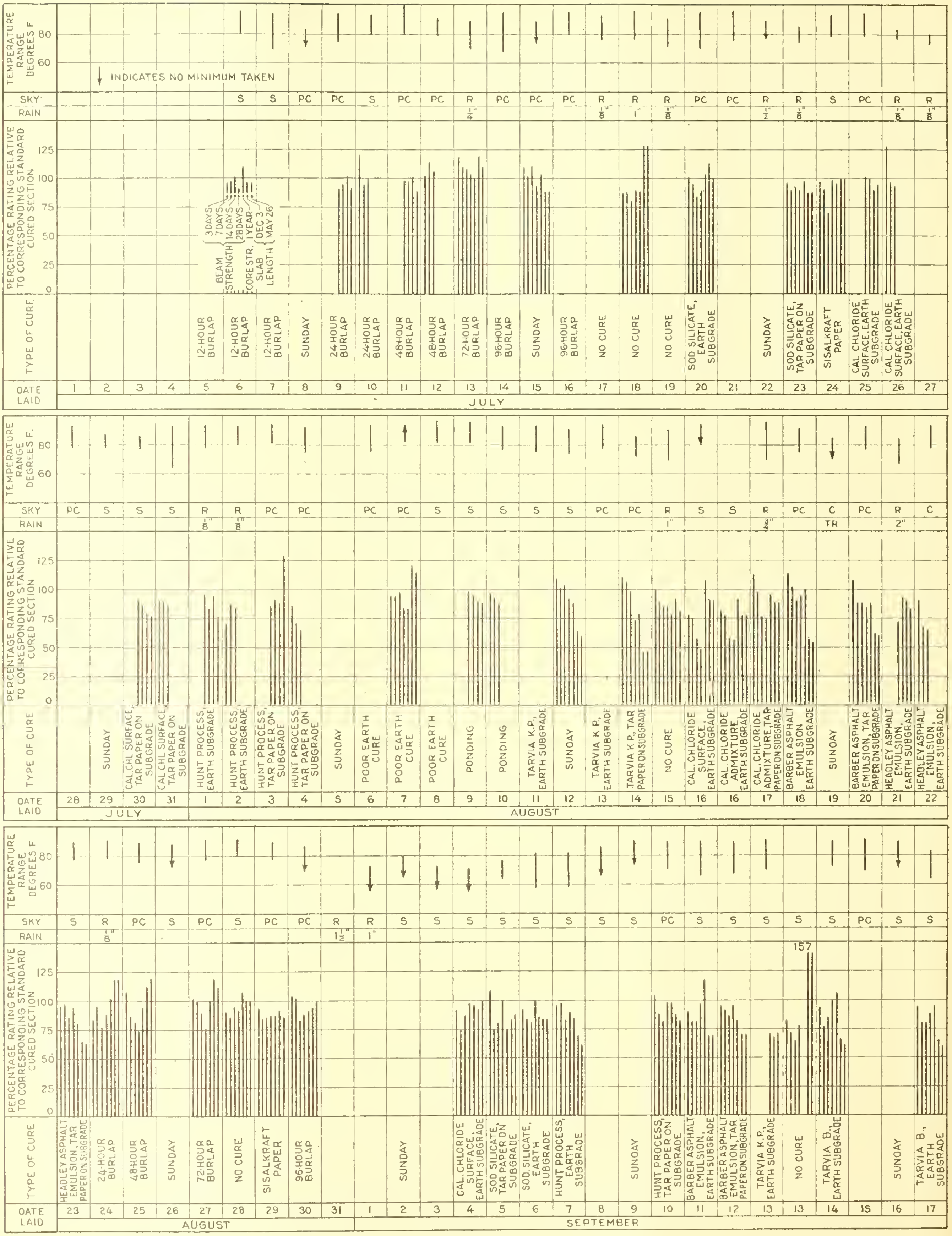


FIGURE 1.—DAILY RECORD OF CURING TESTS, JULY 5 TO SEPTEMBER 17, AND DIAGRAM OF TEST RATINGS FOR EACH SECTION

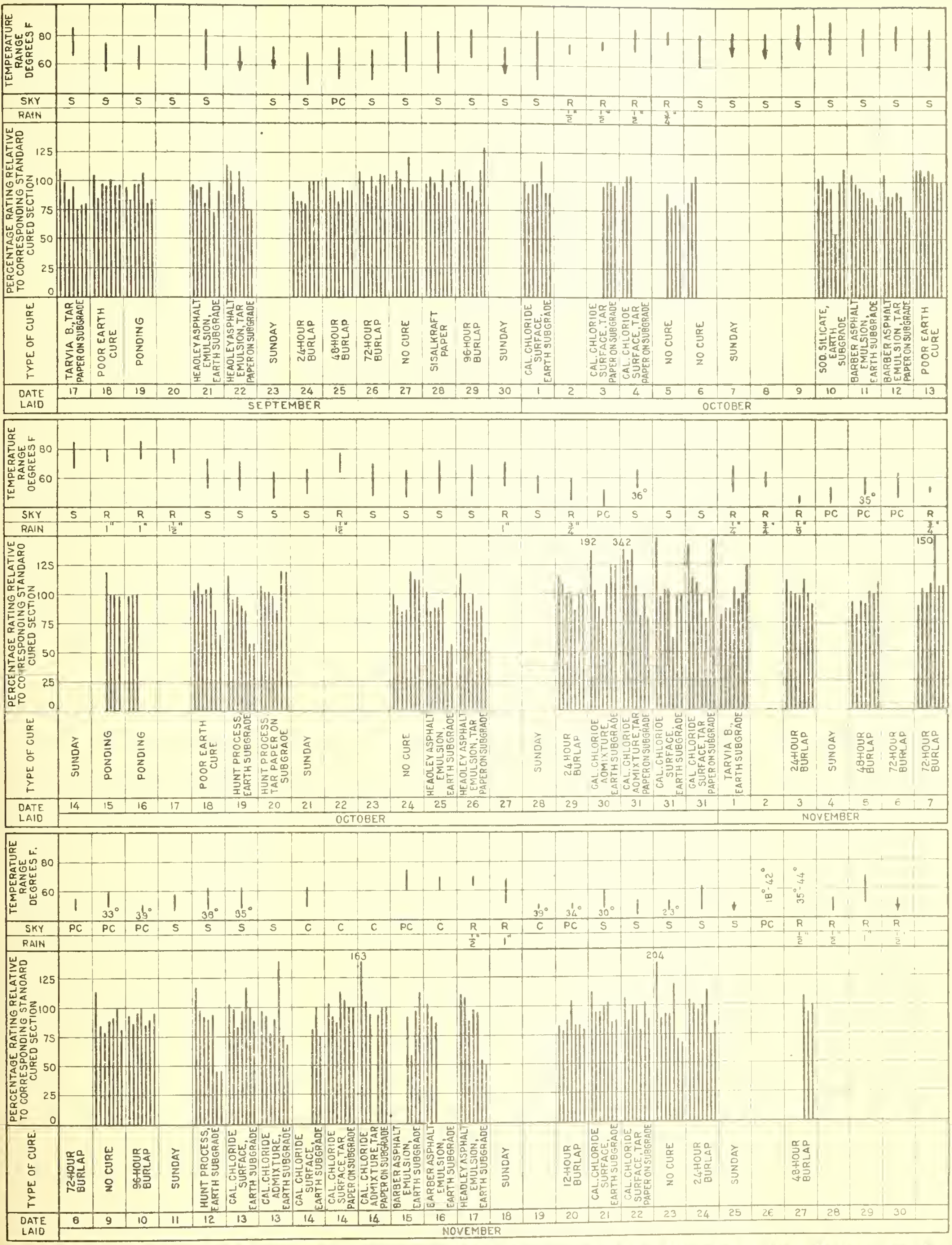


FIGURE 2.—DAILY RECORD OF CURING TESTS, SEPTEMBER 17 TO NOVEMBER 30, AND DIAGRAM OF TEST RATINGS FOR EACH SECTION

function in this respect should certainly be reflected in increased transverse cracking, particularly at the early ages. It is impossible to ignore the evidence supplied by the behavior of the road itself; and the cracking which has developed, although not always explainable, merits careful study. In this connection it should be stated that most of the cracks which have developed to date on this road are due probably to temperature or moisture changes or both, rather than to traffic. This conclusion is based not only on the general appearance and distribution of the cracks, but also on the fact that the pavement does not carry very heavy traffic and, moreover, was constructed on a uniform subgrade, a condition which still further reduces the tendency to failure through overloading.

The efficiency of the various curing methods as regards cracking has been studied by direct comparison of the average lengths in feet of uncracked slab for the various methods as well as by comparison of each method with the corresponding earth and water cured section by means of the percentage rating referred to above. It will be recalled that every effort was made during the construction of the pavement to allow free longitudinal movement of the two sides with respect to each other. The test results indicate, however, that a complete separation was not attained. Reference to the test data given in Table 5 will show that the average spacing between cracks on those standard-cured sections opposite experimental sections which developed excessive cracking was much less than in those cases where only normal cracking developed on the research side. This matter will be discussed in some detail later. It is mentioned here simply to show why it is necessary to compare the actual average uncracked slab lengths for the various cures directly as well as to compare the length of uncracked slab for each experimental section with the corresponding standard cured section. Due to the practical certainty that the slabs were not entirely free along the longitudinal joint, the authors are inclined to believe that a somewhat truer comparison of cracking is obtained by the direct method.

Machine for Laying Burlap Follows the Mixer

The above discussion of the significance of the various methods for evaluating curing efficiency represent the authors' views as to how the data should be interpreted. The reader is of course free to interpret the test results in any way which seems logical to him. In order to permit him to do this, the results of all of the experimental data are presented barring only certain erratic individual beam test results which were eliminated by the method of least squares.

CURING METHODS DISCUSSED IN DETAIL

In order to give adequate consideration to the effect of weather conditions on the results of these tests, it is necessary to study the data obtained from each curing method for each cycle individually before any general comparisons are attempted. The first portion of this discussion, therefore, deals with the various experimental curing methods in order, with special reference to variations in results in the different cycles for the same method and the possible effect of variations in temperature, humidity, and rainfall on the results obtained. Tables 8 to 19, inclusive, give for each curing method a summary of the data presented in Tables 4 and 5 and Figures 1 and 2.

Concrete cured with wet burlap for various periods with no further curing.—A total of 18,810 feet of pavement was cured with wet burlap. This included 2 sections of 12-hour curing, 6 sections of 24-hour curing, 5 sections of 48-hour curing, 4 sections of 72-hour curing, and 4 sections of 96-hour curing. A summary of the data and test ratings for the sections cured in this manner is given in Table 8.

The 2 sections cured with wet burlap for 12 hours had the burlap removed after the end of the 12-hour period, regardless of the time of day. Care was taken to have the burlap thoroughly saturated during the entire 12 hours under cure. One of the two sections cured for 12 hours with burlap was laid July 5 to 7, and the other on November 20. It is interesting to note the low rating values obtained with the section laid November 20. These results emphasize the danger of curing for a short period during cool weather, thus permitting drying out before the concrete has attained sufficient strength to withstand shrinkage stresses. It will be observed by reference to Table 4 that the beam strength at 3 days of the concrete placed on November 20 was only 178 pounds per square inch or less than one-half the beam strength of the corresponding section laid July 5 to 7.

The 6 sections cured for 24 hours with wet burlap show somewhat erratic results as regards relative value. In some cases unusually high core strengths are noted for no apparent reason. In the cycles of August 24 and September 24 the beam strength rating is low. There is the same tendency for excessive cracking to develop when the concrete is cured for a short period at low temperatures, as may be noted by reference to cycle No. 6, laid November 24. Reference to Figure 1 will show that this concrete was exposed to very low temperatures immediately following the placing of the section. A modulus of rupture of only 87 pounds per square inch was attained at three days, although the strength of the concrete at later ages was approximately the same as for other sections laid during warmer weather.

There is nothing of particular significance to note regarding the 48-hour burlap period except to point out that the sections laid in warm weather show less cracking and the one laid on November 27 more cracking as of December 3, than the corresponding standard section. On May 26, 1930, the crack rating of the section laid November 27 had increased to 100 per cent. It will be noted, however, that the average slab length is only 33 feet.

The sections cured for 72 hours with wet burlap give a very good account of themselves, especially as regards cracking. All of the four sections show less cracking than the corresponding standard sections. It should

TABLE 8.—*Summary of data and test ratings for sections cured with wet burlap*

Duration of treatment	Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
Hours			*Feet*	*°F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
12	1	July 5, 6, 7	1,392	93–76	⅜	96	98	101	89	109	53.6	51.6	96.2	96.3
	2	Nov. 20	661	55–35	2½	84	79	88	105	84	50.8	44.1	84.6	80.0
24	1	Aug. 24	1,022	85–78	2½	84	94	77	88	102	63.9	63.9	118.8	118.8
	2	July 9, 10	1,334	94–74	1½	92	96	102	93	122	58.0	58.0	95.6	100.0
	3	Oct. 29	285	60–44	1⅞	115	108	98	101	85	71.2	71.3	100.0	100.0
	4	Sept. 24	671	81–59	1½	92	83	84	82	100	95.9	83.9	100.0	100.0
	5	Nov. 1, 3	778	60–41	¾	113	101	100	98	113	77.8	70.7	100.0	90.9
	6	Nov. 24	917	53–36	2½	106	102	96	100	115	45.8	43.7	75.0	85.7
48	1	Aug. 25	1,005	83–79	2½	107	87	81	74	94	47.9	47.9	114.3	119.0
	2	July 11, 12	804	94–73	2	98	96	101	89	102	57.4	50.2	114.3	106.2
	3	Sept. 25	1,008	82–61	2¼	104	91	92	83	94	77.5	72.0	92.3	92.8
	4	Nov. 5	824	62–45	¾	93	87	94	91	101	82.4	82.4	100.0	110.0
	5	Nov. 27	718	61–46	2					110	35.9	32.6	95.0	100.0
72	1	Aug. 27	1,071	81–70	2½	102	100	90	76	99	53.6	46.6	120.0	113.0
	2	July 13	1,040	93–73	1⅞	117	111	108	104	100	69.3	65.0	120.0	112.5
	3	Sept. 26	867	82–63	2¼	110	100	93	105	96	61.9	57.8	107.1	106.6
	4	Nov. 6, 7, 8	1,161	65–48	1½	90	103	100	109	150	68.3	68.3	105.9	105.9
96	1	Aug. 30	867	81–61	2½	104	102	84	89	92	54.2	54.2	93.7	100.0
	2	July 14, 16	788	92–74	2	110	103	112	93	104	52.5	52.5	86.6	86.7
	3	Sept. 29	607	83–65	2¼	111	100	90	96	83	60.7	60.7	110.0	130.0
	4	Nov. 10	990	64–48	1½	92	87	94	99	85	52.1	52.1	89.4	94.8

be noted, however, that considerable rain fell either immediately after or shortly after the completion of the 72 hours of burlap cure on each of the first three cycles. Cycle No. 4, laid November 6, 7, and 8, received no rain after completion until November 17, six days after the completion of the 72-hour burlap cure and one day before the completion of the cure on the standard side. Analysis of this section is, however, complicated by the fact that three-fourths inch of rain fell on November 7, after 755 feet had been laid. Just what effect this rain had is uncertain. Reference to the detail crack survey indicates about the same number of cracks on the sections laid before and after the rain. It is obvious that any rainfall occurring during the 10-day standard curing period, especially on sections cured with water for only a short time, such as the burlap cures, may influence the relative efficiency of this type of cure to a marked degree.

In contrast with the 72-hour cures, the sections cured for 96 hours gave in general less favorable results than the standard method. However, the differences are not great and may possibly be accounted for by other factors than curing.

In general it may be said that, aside from the obvious tendency for increased cracking to develop on sections cured for short periods under low temperature conditions, the various burlap cures produce results very similar to those obtained by means of the standard earth and water method. As stated above, this conclusion must be qualified by recognition of the probable influence of rain upon the results obtained in this particular experiment. Any conclusions regarding the merits of burlap curing for periods up to 96 hours which may be based on these tests lose much of their significance because of this fact. It is exceedingly unfortunate that the season of 1928 was an exception to the general rule as regards rainfall in this territory and it is still more unfortunate that none of the burlap or "no-cure" sections, which were the ones most probably influenced by rainfall, were laid during the early part of September. No rain at all fell during this month except on the first.

"No-cure" sections.—For purposes of comparison, all artificial curing was omitted from nine sections totaling 7,524 feet, the individual sections varying in length from 322 to 1,259 feet. The word "artificial" is used advisedly, because here again rainfall may and probably did affect the results to a marked degree. Reference to Table 9 and Figure 1 will show that rain fell during the normal 10-day curing period on eight of the nine "no-cure" sections. The one exception is that laid on September 13. No rain fell on the section laid on October 5 and 6 until the ninth day. The rainfall indicated in Figure 1 for October 5 occurred in the early morning before any concrete was laid. The authors believe that it would not be fair to rate as a "no-cure" section any section of pavement upon which rain fell during the normal 10-day curing period, and no such comparisons will be made in this report. Referring then to the section laid on September 13 as one upon which no rain fell, we are confronted immediately with the surprising fact that the "no-cure" side of the pavement developed less cracking than the side cured under earth and water. It would be obviously absurd to conclude from this that no curing gives better results than the standard method and we are consequently forced to search further for a reason why this particular condition should have developed. Refer-

WET BURLAP ON ONE SIDE OF LONGITUDINAL JOINT, EARTH AND WATER CURE ON OTHER SIDE

Table 9.—*Summary of data and test ratings for sections on which no curing method was used*

Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
					3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
		Feet	° *F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
1	Aug. 15	695	90–75	2⅞	100	91	86	86	78	49.6	43.4	92.8	81.3
2	Aug. 28	1,117	81–66	2½	90	85	95	92	108	48.6	48.6	100.0	100.0
3	July 17, 18, 19	1,087	90–75	⅞	88	90	83	92	89	64.0	63.9	129.4	129.4
4	Sept. 27	1,055	82–64	2¼	97	110	101	95	120	48.0	48.0	95.4	95.4
5	Oct. 5, 6	1,259	86–67	2	91	78	80	77	83	74.0	74.0	100.0	105.8
6	Sept. 13	322	82–62	0	84	72	64	78	----------	46.0	46.0	157.1	157.1
7	Oct. 24	598	64–47	2⅞	99	93	86	87	121	85.4	85.4	114.4	114.3
8	Nov. 23	742	55–38	2½	204	90	93	94	120	30.9	28.5	70.8	69.2
9	Nov. 9	649	64–48	1½	113	87	83	88	91	81.1	64.9	100.0	80.0

Table 10.—*Summary of data and test ratings for sections cured with Sisalkraft paper*

Cycle No.	Date laid	Length of cycle	Duration of treatment	Average temperature range 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
		Feet	*Hours*	° *F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
1	Aug. 29	1,031	24	80–61	2½	94	84	86	88	88	41.3	36.8	92.0	85.7
2	July 24	1,008	48	89–76	½	98	93	72	99	97	56.0	56.0	100.0	100.0
3	Sept. 28	924	24	83–65	2¼	98	104	98	90	110	57.8	54.4	93.7	100.0

Sisalkraft Paper on Research Side of Longitudinal Joint

ence to Table 5 shows that the average uncracked slab length on the earth-cured side of this section was only 29.3 feet as compared with an average for the entire nine sections of 53 feet. A search for the reason for this excess cracking developed the fact that this portion of the pavement had been laid on a new fill on the north (or standard-cure) side and that the cracks observed were probably caused by settlement of this fill. It will be noted further that the section is only 322 feet in length. In view of the above condition it is obvious that this section should not be considered.

The section laid on October 5 and 6 remains, therefore, the only one which may possibly be rated as a "no-cure" section. It may be of interest to study this section in some detail. No rain fell until October 15, or nine days after the completion of the section. The average maximum temperature during the first 10 days was 86° F., the average minimum daytime temperature 67° F., and the weather was clear and sunny during the whole period. It will be observed that the section rates very low on both transverse and crushing strength. Average uncracked slab, length, however, is the same as for the standard cure side. This, in view of the temperature and general weather conditions prevailing, is a very good showing.

In general, it is believed that, due to the possible effect of rainfall upon the results, no use should be made of the data obtained from the so-called "no-cure" sections.

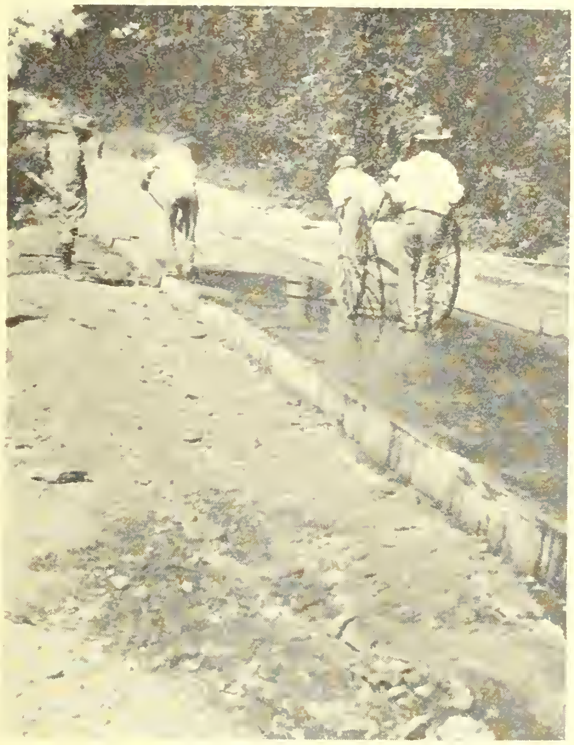
Sisalkraft paper.—On 2,963 feet of pavement laid in three cycles, a reinforced waterproof paper known as Sisalkraft was used in place of wet burlap. In the first and third cycles, the paper remained in place 24 hours; in the second cycle, 48 hours. None of these sections was entirely uninfluenced by rain during the first 10-day period. The one laid July 24 was subjected to the most severe conditions as the record shows that only ¼-inch of rain fell during the first seven days. Reference to Table 10 will show that this section behaved normally from the standpoint of cracking, but that the strength ratings are somewhat low.

Sodium solicate.—A solution of sodium silicate, applied as recommended by the manufacturers, was used on 4,112 feet of pavement, laid in five sections, on two of which tar paper was placed on the subgrade before the placing of the concrete. The data relative to these sections are summarized in Table 11. From the standpoint of severity of curing conditions as evidenced by lack of rain, the sections laid on September 5 and 6 are the most interesting. No rain fell on either of these sections, totaling 2,196 feet, until October 2, almost an entire month. The average maximum temperature during this period was 84° F., the average minimum daytime temperature 61° F., and the number of 100 per cent sunny days was 21. Evidence of the severity of the curing condition is found in the relatively low rating attained by these sections from the standpoint of both strength and cracking. Reference to Table 11 brings out the fact that these sections failed to rate as high as 90 per cent of standard in either core strength or uncracked section length. The data also indicate that the use of tar paper on the subgrade did not affect the results. It will also be noted that the average slab length on the sections laid September 5 and 6 are relatively low, indicating again a lack of curing. Some rain fell on the sections laid July 20 and 23. To just what extent rain may have affected these results is problematical. It is interesting to note, however, that from the standpoint of uncracked slab length, the section laid on July 20 (earth subgrade) is rated 100

Table 11.—*Summary of data and test ratings for sections cured with sodium silicate*

Subgrade	Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
			Feet	*° F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
Earth	1	July 20	823	88–76	⅞	101	96	85	89	103	63.3	54.9	115.4	100.0
	2	Sept. 6	1,087	89–68	0	93	84	81	100	86	40.3	40.2	85.2	85.2
	3	Oct. 10	305	80–62	3½	102	106	95	95	53	43.6	38.1	100.0	112.5
Tar paper	1	July 23	788	89–76	½	97	92	93	91	97	49.2	43.8	87.4	88.9
	2	Sept. 5	1,109	88–67	0	108	76	81	100	75	41.4	42.6	80.0	88.5

per cent of standard on May 26, whereas the section laid on July 23 (tar-paper subgrade) shows only 89 per cent of standard. The section laid July 20, it will be noted, received a ½-inch rainfall on July 22, whereas the section laid on July 23 received only very slight rainfall on the 26th and 27th.



Applying Sodium Silicate to Surface of Pavement

Calcium chloride surface application.—Thirteen sections of pavement, totaling 9,746 feet in length, were cured with a surface application of calcium chloride, applied at the rate of 2 pounds to the square yard. Individual sections varied in length from 219 to 1,635 feet. Table 12 gives in summarized form the results obtained from these tests. Here again the most interesting section from the standpoint of rainfall is the one laid on September 4. This section was subject to the same weather conditions as the two sodium silicate sections mentioned above. The section is rated less than 100 per cent from the standpoint of strength. The uncracked section length rating was 93 per cent on December 3 but rose to 100 per cent on May 26, due to the fact that two additional cracks had developed on the standard side to only one on the research side. It should be noted, however, that the average length of uncracked slab on this section was only 39.2 feet on May 26. This may be compared to an average of 48.8 feet for all eight sections laid on earth subgrade. (See Table 5.) The length of the corresponding standard-cured side, moreover, was only 39.2 feet as against an average of 53.3 feet for the eight sections. This condition illustrates the possibility of restraint along the longitudinal joint and its influence on the amount of cracking developing on the standard side.

Spreading Calcium Chloride on Pavement

In direct contrast with this section, we may note the section laid on October 1. Although the uncracked slab rating for this section is only 89 per cent, the average length of slab on May 26 was 86.5 feet, or more than two times the length of slab on the section laid September 4. It will be noted that over 2 inches of rain fell on this section during the three days immediately following its construction. On the other hand, the section laid on November 21 shows relatively short crack spacing. This section, it will be observed, was laid during very cold weather, the temperature going below freezing on the second day. It is possible that this fact, coupled with the possibility that drying out took place between November 21 and November 27, the date of the first rain following the laying of the section, accounts for the relatively large number of cracks observed. On October 31, a short section (198 feet) was laid which showed an average uncracked slab length of 66 feet on May 26. This section was laid in somewhat warmer weather than the one placed on November 21 and, moreover, received considerable rainfall for three days following its placing.

In regard to the sections laid on tar paper, it may be of interest to discuss the possible reasons for the relatively large amount of cracking which has taken place on the section laid on July 30 and 31, as compared with most of those laid later in the year. Reference to the notes taken during construction indicate that great difficulty was experienced in obtaining sufficient water for curing just at this time. This section was laid at the beginning of a new set-up at the extreme west end of the project in Shelby County, and it was necessary to pump water a distance of approximately four miles.

Table 12.—*Summary of data and test ratings for sections cured by surface application of calcium chloride*

Subgrade	Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
			Feet	*°F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
Earth	1	Aug. 16	330	90–75	2⅞	77	74	58	49	107	30.0	27.5	90.9	91.6
	2	July 25, 26	1,635	90–76	⅜	101	100	93	96	127	56.4	51.1	96.6	93.7
	3	Oct. 1	779	84–68	2¼	100	88	97	98	117	86.6	86.5	88.9	88.9
	4	Sept. 4	588	87–66	0	92	74	87	97	96	42.0	39.2	92.8	100.0
	5	Oct. 31	198	59–44	1⅞	147	99	104	105	64	66.0	66.0	100.0	100.0
	6	Nov. 21	891	54–36	2½	114	95	95	100	103	40.5	37.1	86.4	87.5
	7	Nov. 14	219	62–45	1½					80	73.0	54.8	100.0	75.0
	8	Nov. 13	625	63–45	1½	102	99	82	96	119	78.1	69.4	100.0	88.9
Tar paper	1	July 30, 31	1,139	93–79	¼	92	87	80	77	92	34.5	31.6	90.9	86.2
	2	Oct. 3, 4	1,178	86–66	¾	91	100	100	96	94	69.3	69.3	105.9	105.8
	3	Oct. 31	493	59–44	1⅞	143	115	110	100	80	70.4	54.8	100.0	122.2
	4	Nov. 22	966	54–35	2½	109	87	100	100	80	48.3	38.6	105.0	88.0
	5	Nov. 14	705	62–45	1½	103	91	86	114	107	88.1	88.2	100.0	100.0

Appearance of Freshly Spread Calcium Chloride

The inspector's notes show that extreme difficulty was experienced in obtaining sufficient water for burlap curing and it is possible that this fact may account in part for the relatively large amount of cracking. It is interesting to note also that this is the only calcium chloride section on which scaling was observed. A small amount of thin scale has developed at the west end of the section.

In Figure 3 the average slab lengths on the sections cured with sodium silicate and calcium chloride (surface application) have been plotted against the total rainfall in inches on the sections during the normal 10-day curing period. An inspection of this figure indicates that rainfall may possibly have influenced the behavior of certain of these sections to a considerable extent. It is significant that in every case when the rainfall was one-half inch or less the distance between cracks was relatively short. Only one of the six sections showed a spacing as high as 50 feet. Conversely, eight out of the 12 sections upon which more than one-half inch rain fell showed average uncracked slab lengths of 55 feet or more. Of the other four, two were laid in very cold weather (November 21 and 22). The section laid on August 16 also shows a relatively large amount of cracking. Here again reference to the construction notes indicates trouble with the water supply and it is possible that this section also received an inadequate burlap cure prior to the application of the calcium chloride.

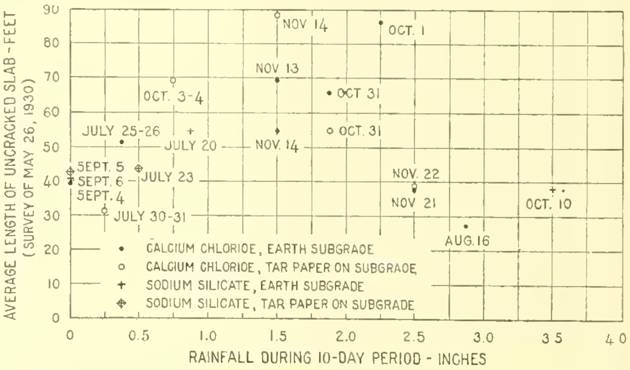


Figure 3.—Relation Between Rainfall During the 10-Day Curing Period and Average Length of Uncracked Slab for Sections Cured by Means of Surface Applications of Calcium Chloride and of Sodium Silicate

Calcium chloride admixture.—A calcium chloride admixture in the proportion of 2 pounds to a bag of cement was used in the concrete on six sections of this project, totaling 1,818 feet. Three of the sections were laid on earth subgrade and three on tar paper. Inasmuch as it was impossible to lay concrete containing an admixture on one side only, the practice was followed, in the case of the calcium chloride admixture, of laying the experimental pavement for the full width and comparing the behavior of this full width section with the adjacent earth and water cured half-section laid on the same day. The data regarding these sections are summarized in Table 13. In so far as beam strength in concerned, four of the sections show the characteristic high early (3-day) strength of concrete in which calcium chloride is used as an admixture, but in all cases the strength ratio, as compared to the earth-cured beams, rapidly falls off at the later periods. In every case but one the rating at 28 days is considerably less than 100 per cent. Core strengths for the individual cycles are somewhat erratic. With one exception, however, the crushing strength rating is above 90 per cent of standard cure.

The spacing of cracks in these sections also varies widely, with respect to both average length and ratio to the corresponding earth-cured sections. The sections laid on August 16 and 17 both show less than 90

Table 13.—*Summary of data and test ratings for sections cured by admixture of calcium chloride*

Subgrade	Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
			Feet	*° F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
Earth	1	Aug. 16	266	90–75	2⅞	82	76	59	56	91	29.6	29.6	77.8	77.8
	2	Oct. 30	507	60–44	1⅞	192	104	90	79	108	63.4	63.4	125.0	125.0
	3	Nov. 13	280	62–45	1½	97	93	77	91	138	35.0	31.1	75.0	66.7
Tar paper	1	Aug. 17	338	89–76	2⅛	113	98	77	75	96	37.6	37.6	88.9	88.9
	2	Oct. 31	227	59–44	1⅞	342	129	139	106	83	56.7	45.4	100.0	80.0
	3	Nov. 14	200	62–45	1½	163	105	95	----------	94	100.0	100.0	100.0	100.0

Table 14.—*Summary of data and test ratings for sections cured by Hunt Process*

Subgrade	Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
			Feet	*° F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
Earth	1	Aug. 1, 2	1,459	92–78	0	97	84	95	78	73	31.0	28.6	89.4	86.4
	2	Sept. 7	800	89–69	0	96	98	83	90	86	26.7	23.5	70.0	61.8
	3	Oct. 19	336	70–50	3¼	116	95	98	91	88	22.4	22.4	60.0	60.0
	4	Nov. 12	1,073	63–46	1½	118	97	91	90	94	21.5	19.5	46.0	45.5
Tar paper	1	Aug. 3, 4	1,367	92–77	0	86	93	89	132	86	27.3	24.4	72.0	66.1
	2	Sept. 10	975	86–65	0	105	90	83	99	99	29.5	25.6	87.9	84.2
	3	Oct. 20	482	68–49	3¼	107	103	103	99	86	48.2	48.2	120.0	120.0

per cent rating as compared with earth cure. Average spacing of cracks in both cases is less than 40 feet. Both of these sections were laid during a period of acute water shortage. This fact is noted and commented upon, not as a definite reason for the excess cracking during this period, but simply as a possible reason for the condition observed.

On November 13 and 14 two short sections were laid which are interesting in that the section laid on earth subgrade (November 13) showed an average uncracked slab length of only 31 feet, whereas the one laid on tar paper showed an average of 100 feet. There is nothing in the record which would account for this wide difference. It should be noted, however, that both of these sections were very short (less than 300 feet) so that the possibility of accidental variables affecting the results is considerably increased.

Hunt Process.—Seven sections totaling 6,492 feet were cured by means of the so-called Hunt Process method, which consists in spraying specially prepared cut-back asphalt material upon the surface of the concrete at about the same interval after placing as burlap would ordinarily be applied. Burlap as well as all subsequent curing is eliminated by this method.

By reference to Table 14 and Figures 1 and 2 it will be observed that in so far as beam strength is concerned the modulus of rupture at various ages up to 28 days is in general in excess of 90 per cent of that for the corresponding earth cured specimens. There is also noted a tendency for the ratio to fall off at later periods which may be accounted for by the fact that the specimens were not protected on the sides by the seal coat. However, as has been already noted, this matter was investigated during the third cycle by coating the sides as well as the top of the specimens. Reference to Table 7 will show no consistent trend in either direction as the result of this treatment. As to core strength, all of the sections cured by Hunt Process show less than 100 per cent of the strength shown by the standard-cured sections. These data would indicate that this method of cure results in a somewhat lower crushing strength of the concrete than is attained by the standard earth and water method.

Referring now to the cracking which has developed on the sections cured with Hunt Process, we note a condition which appears to be common to practically all of the sections cured with asphalt or tar. Table 14 shows that transverse crack spacing on these seven sections averaged only about 25 feet. This is far less than the average spacing for any of the cures which do not involve the application of asphalt or tar and is also much less than that occurring in the corresponding earth-cured sections. The section ratings as regards cracking range from 45.5 to 86.4 for sections laid on earth and from 66.1 to 120.0 for sections laid on tar paper. The section laid on tar paper on October 20 is the only one which shows more cracking on the standard-cured side. There is nothing to indicate why this section should have behaved in a manner so different from the one laid the day before (October 19) on earth subgrade. Both sections are comparatively short (less than 500 feet), so that the presence of one crack more or less has a much larger influence on the section rating than in the case of longer sections.

Barber asphalt emulsion (*Curcrete*).—A total of 6,897 feet of pavement, in seven sections, was cured by means of a spray application of curcrete, an asphaltic emulsion manufactured for the purpose.

First discussing beam strength, we find on reference to Table 15 and Figure 1 much the same trend as in the case of Hunt Process. With only one or two exceptions the strength ratings are less than 100 per cent of earth cure, with the same tendency to lower ratings at the later ages. In the third cycle on earth subgrade as well as in the third cycle with tar paper on the subgrade, a double set of beams was cured for the purpose of ascertaining the effect of leaving the sides unprotected. This matter has previously been discussed and is mentioned here simply to indicate that, in this particular case, the same tendency to lower relative strengths with increase in age was noted for the specimens cured on both sides and top as in the case of the specimens cured on the top only. (See Table 15.) It would seem therefore that, in so far as it is possible for specimens of this type to indicate concrete strength, the method of procedure used in these tests was as

TABLE 15.—*Summary of data and test ratings for sections cured with Barber asphalt emulsion*

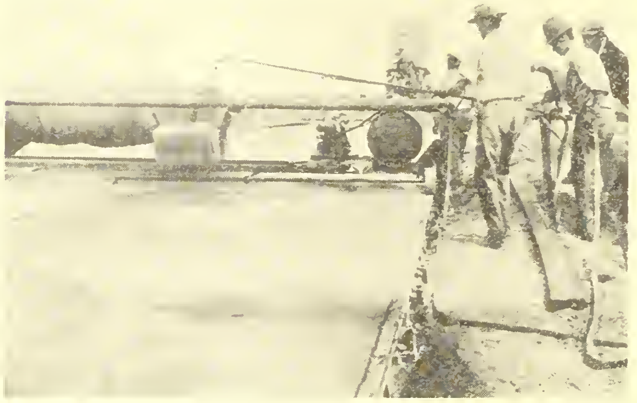
Subgrade	Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings [1]				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
			Feet	*°F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
Earth	1	Aug. 18	1,016	89–76	2⅛	115	101	90	95	98	21.6	19.6	57.4	53.8
	2	Sept. 11	1,097	86–64	0	92	83	82	98	119	21.5	20.3	70.6	70.4
	3	Oct. 11	1,003	79–61	3½	107	99	95	92	86	35.8	33.4	85.7	80.0
						100	100	97	92					
	4	Nov. 15, 16	1,021	56–38	1½	92	58	97	112	103	46.4	44.4	90.9	87.0
Tar paper	1	Aug. 20	1,003	89–76	2⅛	107	87	88	83	88	25.1	22.3	62.5	60.0
	2	Sept. 12	1,063	84–63	0	97	93	88	96	82	25.9	23.6	70.7	71.2
	3	Oct. 12	694	78–60	5	106	87	88	92	88	36.5	34.7	73.6	70.0
						100	94	99	92					

[1] Where two sets of results are shown for a given section the figures given on the upper line represent tests of beams coated on top only, while those on the lower line represent tests of beams coated on sides as well as top.

TABLE 16.—*Summary of data and test ratings for sections cured with Headley asphalt emulsion*

Subgrade	Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings [1]				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
			Feet	*° F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
Earth	1	Aug. 21, 22	1,085	87–78	2⅝	72	93	90	82	91	20.9	20.1	67.3	64.8
	2	Sept. 21	909	79–52	0	97	91	93	81	99	34.9	34.9	73.1	92.4
	3	Oct. 25	1 004	62–46	2⅞	101	86	89	89	97	24.5	22.8	51.2	56.8
						88	94	76	78					
	4	Nov. 17	485	53–35	1½	111	109	87	97	95	25.5	24.2	52.6	50.0
Tar paper	1	Aug. 23	1,050	87–78	2⅝	94	98	86	94	18	22.8	20.6	65.2	62.8
	2	Sept. 22	773	79–54	½	116	111	89	112	96	38.6	36.8	75.0	76.2
	3	Oct. 26	531	61–45	2⅞	118	101	94	99	86	53.1	37.9	90.0	64.2
						140	94	95	104					

[1] Where two sets of results are shown for a given section, the figures given on the upper line represent tests of beam coated on top only, while those on the lower line represent tests of beam coated on sides as well as top.



SPREADING BITUMINOUS MATERIAL ON RESEARCH SIDE OF PAVEMENT

satisfactory as it was possible to devise. The core strength ratings for these sections parallel quite closely the corresponding values for Hunt Process. Only in one case, that of cycle No. 2, earth subgrade, was the crushing strength of the concrete appreciably higher. It so happens that this particular cycle received no rain for three weeks. However, the corresponding section on tar paper subgrade laid on the following day shows a core strength rating of only 82 per cent of standard. In general, the same conclusion may be drawn as in the case of Hunt Process: That the crushing strength of the concrete is somewhat reduced by the use of this curing method.

The same tendency to excess transverse cracking is noted on all of the seven sections cured with Curcrete, as was shown in the case of Hunt Process. The individual section ratings vary from 54 to 87 per cent of standard. The average uncracked slab length as of May 26 is about 25 feet. This figure is in striking contrast with the average of about 55 feet for the 21 burlap cured sections. It is of interest to note that two of the sections showing the greatest cracking were laid on September 11 and 12. These sections received no rain at all for a period of three weeks. It is also notable that the section laid on August 18 showed excessive cracking in spite of a two-inch rainfall on August 21. Another point worthy of note is that cycle No. 4, laid November 15 and 16, showed a crack spacing of 44.4 feet, considerably greater than that of any of the sections laid earlier in the year. Reference to Figure 1 will show that temperatures following the laying of this section were rather low.

Headley asphalt emulsion.—This asphaltic emulsion was used on seven sections for a total of 5,837 feet. The essential data are given in Table 16. The same general tendencies as regards beam and core strength apply to these sections as to those cured by Hunt Process and by Curcrete. There are no particularly unusual results and it is believed that the same general conclusions regarding strength may be drawn.

Similar comments may be made regarding cracking. The average rating for the seven sections is approximately 65 per cent of standard and the average uncracked slab length is about 26 feet. The sections laid September 21 and 22 show less cracking than those laid August 21 and 23 and also less than the section laid November 16 and 17. The section laid October 26 also shows relatively few cracks, although the section rating is low (64 per cent).

Tarvia K. P. and Tarvia B.—Three sections were cured by means of a spray application of Tarvia K.P. This is a cut-back tar product used in cold patch work, having a specific viscosity (Engler, at 40° C.) of 78.4. The results obtained from these tests are summarized in Table 17. One of these sections, laid on September 13, was only 259 feet long and it is reported that, on this section, the spraying was very unsatisfactory. No beam strengths are available for this section and the core strengths are low. The crack rating is higher than for any other of these sections, but the uncracked slab length is only 14.4 feet. Both the remaining sections show characteristics very similar to those exhibited by the sections cured with asphalt. The same

TABLE 17.—*Summary of data and test ratings for sections cured with Tarvia K. P. and Tarvia B*

Type of treatment	Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings[1]				Core strength rating at one year	Average slab length		Slab length rating	
						3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
			Feet	*Degrees F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
Tarvia K. P., earth, subgrade	1	Aug. 11, 13	1,551.5	90–74	3¾	111	103	104	93	88	26.3	21.2	64.4	60.9
Do	2	Sept. 13	259	82–62	0	------	------	------		72	16.2	14.4	68.7	72.2
Tarvia K. P., tar paper on subgrade	1	Aug. 14	816	90–74	3⅞	110	106	98	74	79	18.5	17.0	45.4	45.8
Tarvia B, earth subgrade	1	Sept. 17	440	77–52	0	95	80	80	90	96	22.0	21.0	65.0	61.9
Do	2	Sept. 14	371	80–57	0	94	78	86	100	107	17.7	15.5	66.7	62.5
Do	3	Nov. 1	270	58–44	1⅝	83	90	89	106	95	67.5	67.5	100.0	125.0
						94	100	87	98					
Tarvia B, tar paper on subgrade	1	Sept. 17	719	77–52	0	111	99	84	88	75	37.8	36.0	78.9	80.0

[1] Where two sets of results are shown for a given section the figures given on the upper line represent tests of beams coated on top only, while those on the lower line represent tests of beams coated on sides as well as top.

strength tendencies are noted and also the same excess cracking. The section laid August 11 and 13 had an uncracked slab length rating of 60.9 per cent and an average slab length of 24.2 feet. For the corresponding section laid on tar paper these values become 45.8 per cent and 17 feet.

Tarvia B was applied in four short sections. This is a tar product used in the cold surface treatment of roads. The data for these sections are also given in Table 17. The material used had a specific viscosity (Engler, at 40° C.) of 17.7. Tarvia B gave essentially the same results as the asphalt cures, although the sections are in reality too short to permit of trustworthy analysis. The only section which is out of line with the general results is the one laid November 1, which showed relatively wide crack spacing. This section is only 270 feet in length. Considerable rain fell on the section during the first few days and this fact, together with the low temperature immediately following the placing of the section, probably accounts for the favorable result.

GENERAL COMMENTS REGARDING CRACKING ON ASPHALT AND TAR-CURED SECTIONS

The most reasonable theory to account for the relatively large number of cracks which have developed on the sections cured with asphalt and tar is that the absorption of heat by the black surface followed by cooling at night produced a greater temperature range in slabs cured in this manner than in those cured by the standard earth and water method and that this increased temperature change in turn caused excessive cracking. Measurements conducted by the Bureau of Public Roads and reported in Public Roads for February, 1930[1] indicate that, for comparable conditions of exposure, concrete coated with bituminous material may show a temperature 10° C. higher than uncoated concrete. Evidence that this excess cracking is a temperature effect is also indicated by the fact that considerably more cracking developed on those sections placed in the morning and immediately sprayed with bituminous material than on that portion of the pavement placed in the afternoon. The concrete placed before noon was of course exposed to the sun for a considerably longer period, so that when cooling took place at night a greater drop in temperature occurred. This condition was noted repeatedly during these tests. A typical illustration will be cited.

The Hunt Process section laid on tar paper on September 10 was subjected to practically 100 per cent sunlight and high temperature during the first three weeks of its life. The section is 975 feet in length. The joints were 200 feet apart with the exception of the last section, which measured 175 feet. Approximately the same length of pavement was laid in the forenoon as in the afternoon. On September 11, the day after the concrete was laid, five cracks had appeared in the section laid in the forenoon and none in that laid in the afternoon. On November 9, 1928, a survey of this section showed 14 cracks in the forenoon work and only 8 in the afternoon work. Furthermore, of the 14 morning cracks, 9 appeared in the first 200-foot slab, or more than in the entire afternoon run of approximately 500 feet. On May 26, 1930, the cracks had increased to 20 in the morning work and to 14 in the afternoon, with 12 cracks showing in the first 200 feet.

LAYING TAR PAPER ON SUBGRADE

This section is cited simply as an illustration of a condition which is typical of most of the sections cured with bituminous materials. Unfortunately the crack spacing at early periods was not determined for the sections laid in November, so we are unable to say just how these would compare with those laid in warmer weather. In so far as final cracking is concerned, reference to Table 5 will show in general a greater crack spacing for those sections laid under cold weather conditions. The following examples may be noted: Hunt Process, October 20, 48.2 feet; Barber Curcrete, October 11, 33.4 feet; Barber Curcrete, November 15–16, 44.4 feet; Barber Curcrete, October 12, 34.7 feet; Headley emulsion, October 26, 37.9 feet; Tarvia B, November 1, 67.5 feet. Certain other sections showed excessive cracking even in the cooler weather, such as Hunt Process, November 12, 19.5 and Headley emulsion, October 25, 22.8 and November 17, 24.2 feet.

In general, it is believed that the temperature effects of such bituminous cures as were used in this experiment are sufficiently severe to indicate that these methods are unsatisfactory under the general climatic

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[1] "The Arlington Curing Experiments," by L. W. Teller and H. L. Bosley

Table 18.—*Summary of data and test ratings for sections cured by "poor earth" method*

Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
					3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
		Feet	°*F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
1	Aug. 6, 7, 8	1, 280	92–75	1¾	91	91	97	84	83	55. 6	47. 4	121. 8	114. 7
2	Sept. 18	1, 121	78–52	0	106	86	96	95	101	70. 1	70. 2	93. 7	93. 8
3	Oct. 13	346	76–59	5	110	110	105	110	107	86. 5	86. 5	100. 0	100. 0
4	Oct. 18	571	71–50	2½	103	110	100	104	105	81. 6	63. 5	85. 7	66. 7

Table 19.—*Summary of data and test ratings for sections cured by ponding*

Cycle No.	Date laid	Length of cycle	Average temperature range, 10-day period	Rainfall, 10-day period	Beam strength ratings				Core strength rating at one year	Average slab length		Slab length rating	
					3-day	7-day	14-day	28-day		Dec. 3, 1929	May 26, 1930	Dec. 3, 1929	May 26, 1930
		Feet	°*F.*	*Inches*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Per cent*	*Feet*	*Feet*	*Per cent*	*Per cent*
1	Aug. 9, 10	570	91–74	1¾	99	95	90	89	97	40. 7	38. 0	92. 8	86. 7
2	Sept. 19	1, 330	79–53	0	94	83	96	96	108	63. 4	55. 4	80. 9	83. 3
3	Oct. 15, 16	305	73–52	3	119	100	100	98	98	61. 0	61. 0	100. 0	100. 0

conditions which obtained during these tests. Furthermore, the data seem to indicate that somewhat lower crushing strength results when the bituminous seals are used.

It has been claimed that the relatively poor showing of the bituminous cures is due to the fact that the edges of the pavement were not sprayed with the bituminous material immediately upon removal of the forms. It should be pointed out in this connection that no criticism of the method used was advanced by any representative of an interested company during construction, in spite of the fact that such representatives were present. It seems hardly possible that failure simply to spray the edge of the pavement could have permitted sufficient drying to cause the excessive cracking noted. The area of the outside edge in comparison with the total volume of concrete is so small as almost to preclude the possibility of such action. Furthermore, the manufacturers of bituminous curing materials claim that much of the effectiveness of their material is lost if it is not sprayed on the concrete before final set takes place. This obviously is impossible in the case of the edge, which can not be sprayed until the next day. Moreover the same excessive cracking has been observed in other concrete pavement construction in which the edges of the pavement were treated with bituminous material upon removal of the side forms.

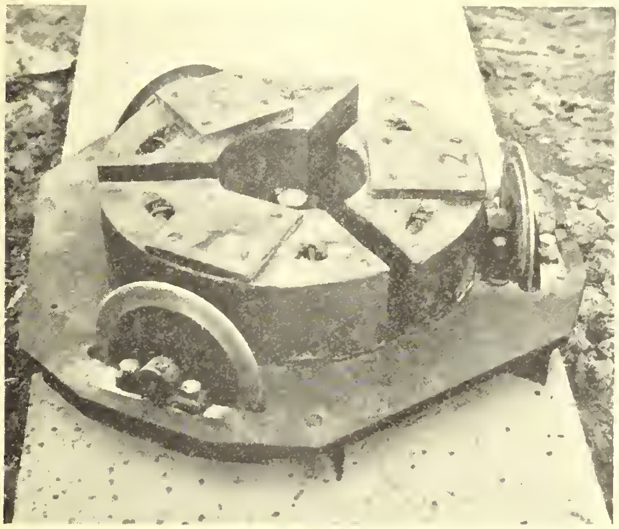
Poor earth cure.—Four sections of pavement, totaling 3,318 feet, were cured by means of earth kept only moist in an effort to simulate what might be termed "poor earth curing." The results are summarized in Table 18. The section laid September 18 is of interest because of the fact that no rain fell until October 2, or fourteen days after placing. This section is rated above 100 per cent in core strength and above 90 per cent in uncracked section length. The average slab length on May 26, 1930, was 70.2 feet. This may be considered an excellent showing. The section laid August 6 to 8 also showed up fairly well from the standpoint of cracking, although low as regards core strength. No rain fell on this section until August 15, seven days after laying. The sections laid October 13 and October 18 are of no great significance, due to the rain which fell during the 10-day period. In general, the sections cured in this manner gave results which compare very favorably with the standard method.

Ponding on One Side, Standard Earth and Water Cure on the Other

Ponding.—On three sections totaling 2,205 feet, the surface of the concrete was cured by ponding in the usual manner. Table 19 gives the essential data regarding these sections. The section laid September 19 is the most interesting due to the fact that no rain fell until October 2, or 13 days. This is a section comparable with that laid on September 18 and cured by the "poor earth" method. The results are somewhat similar, although the crack rating on the ponded section is considerably lower than on the "poor earth" section. The average uncracked slab length on May 26 is also considerably less, amounting to only 55.5 as compared to 70.2 feet for the "poor earth cure." These results are, of course, directly opposed to accepted engineering thought on this subject, which has always held that ponding constitutes the most efficient method of cure possible to apply. No explanation is offered except the possibility that the blanket of dry earth used on the "poor earth cure" section may have acted as an insulating agent, so that the total range in temperature of the concrete was reduced beyond that which was attained by the ponded slab. There are, of course, no test data upon which such a conclusion could be based. The theory is simply noted as a possible explanation for an observed condition which can be explained in no other way. It is certainly true that extreme temperature variation in concrete slabs should be prevented as much as possible especially during the early periods, when it is also essential to retain moisture for hydration. Investigations should be conducted for the purpose of determining the insulating effect of dry earth and dry straw on new concrete, compared to wet earth, wet straw, or ponding.

RESULTS OF SURFACE HARDNESS TESTS DISCUSSED

The tests for surface hardness, using the apparatus developed by the Bureau of Public Roads, were run on selected sections for each of the curing methods. The test, which is fully described in the July, 1929, issue of Public Roads, consists essentially of subjecting the surface of the pavement to the wear produced by three solid steel wheels, 8 inches in diameter and one-fourth inch wide, traveling in a circle 21 inches in diameter. The wheels are mounted on a flat horizontal plate which is operated by a vertical spindle driven by an electric motor. The total weight on the wheels is approximately 450 pounds, and the plate turns at the rate of approximately 35 revolutions per minute. The action of the test is to cut a circular groove in the mortar surface, the depth of which is measured by means of an Ames dial, suitably mounted. Readings are taken after any given number of revolutions of the spindle.



THE MACHINE FOR TESTING SURFACE HARDNESS

The results of the hardness tests, which are given in Table 6, are repeated in graphic form in Figure 3. Depths of wear were measured after 1,000 revolutions, and are recorded in decimals of an inch. It will be noted that the maximum wear in any case was 0.077 inch (or less than one-tenth inch) so that the test is in reality a test of the hardness of the mortar top. Any coarse aggregate appearing during the test would obviously affect the results considerably, especially if the particles were appreciably harder than the mortar, which was the case in this concrete.

In Table 6 the hardness rating is reported as the percentage which the wear on the research side bears to the wear on the standard side, so that values less than 100 indicate harder concrete on the research side and values more than 100 indicate softer concrete on the research side. In making the comparisons at any given location, great care was exercised to select two portions of the surface which were, so far as visual appearance is concerned, identical. This was done in order to eliminate as far as possible variation due to method of finishing, water content, etc. Each value reported in Table 6 represents the average of three tests taken at different locations on the section indicated.

Referring now to the table as well as to Figure 4, on which the results are shown graphically, and considering first the burlap cures, we observe that, in all cases, somewhat less wear was obtained with the experimental method. Tests on the burlap cured sections were all run on sections placed during the first

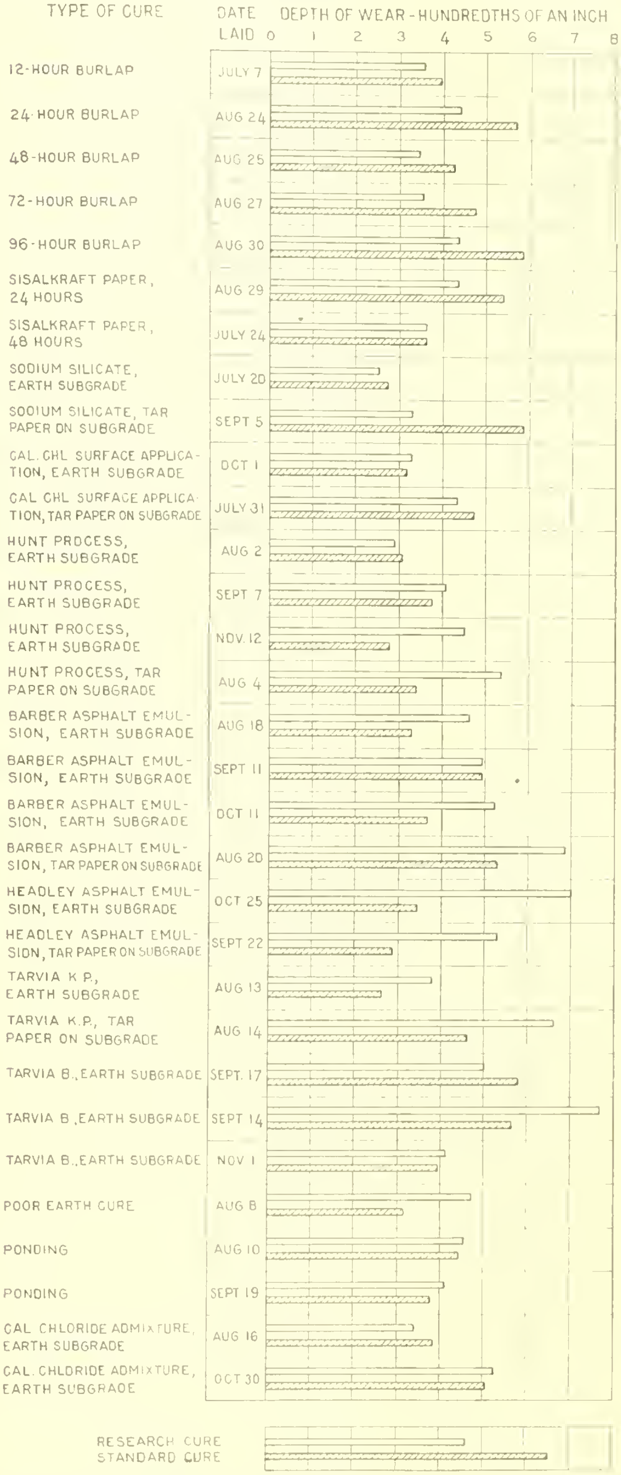


FIGURE 4.—RESULTS OF HARDNESS TESTS OF SECTIONS CURED BY VARIOUS METHODS, COMPARED WITH THE RESULTS OBTAINED FOR THE CORRESPONDING STANDARD-CURED SECTIONS

cycle and for this reason the results are obscured somewhat by the effects of rainfall. The 12-hour section was unquestionably the most severe condition from the standpoint of lack of moisture. This section showed 90 per cent of the wear of the corresponding earth and water cured section, indicating very little difference in surface hardness between the burlap and the standard method. The same comments apply to the concrete cured with sisalkraft paper.

Tests on the section laid September 5 and cured with sodium silicate are interesting. It will be recalled that this section received no rain for a month after laying. The wear on the standard, or earth-cured side, was almost double that on the side cured with silicate. This indicates a definitely harder mortar surface in the case of the silicate cure.

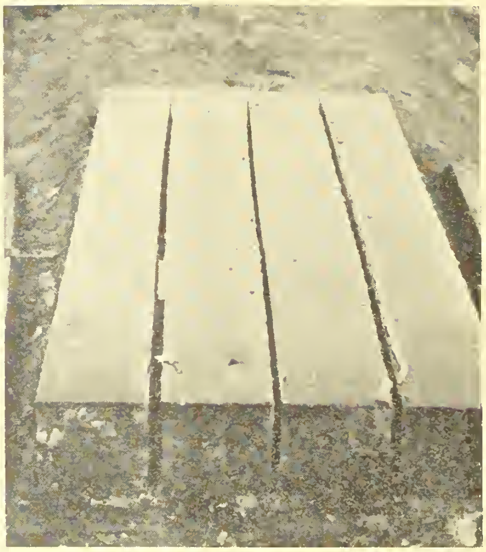
Making Surface Hardness Tests on the Tennessee Curing Project

Tests on sections cured with a surface application of calcium chloride indicate no difference in the hardness of the concrete as the result of this treatment. Essentially the same results were secured on both sections. In the case of the admixture, somewhat less wear was obtained on one of the sections tested (cycle No. 1, earth subgrade), whereas on the other no appreciable difference is noted.

The results of hardness tests on the sections cured with bituminous materials are significant in that a distinct softening of the surface is noted in many cases. On 12 of the 15 bituminous-cured sections on which wear tests were conducted, the depth of wear was greater on the research side. On 10 of these sections the wear was appreciably greater, whereas in only one case was it appreciably less. It is believed that the data indicate definitely that the wear resistance of concrete is not increased and is probably decreased by the use of these bituminous curing methods.

RESEARCH-CURED BEAMS SHOW HIGHER STRENGTH THAN THOSE NOT CURED

In Table 4 are given, for each test section, the results of beam tests made on specimens which were given no curing treatment of any kind, together with the ratios, in each case, of the research-cure specimens to the corresponding no-cure specimens. These data are of interest in showing the efficiency of the several research cures as affecting beam strength under various weather conditions. It will be noted that in general the research-cured beams show substantially higher strength than beams which were not cured. No detailed analysis of these data will be given in this report. The data are made available, however, in case the reader desires to make such an analysis.



Beams Treated With Asphalt Emulsion

GENERAL DISCUSSION

In order to obtain a general idea of the behavior of the several curing methods under average weather conditions, the average values of beam and core strength and uncracked slab length for each curing method have been plotted in Figures 5 to 9, inclusive. Figure 5 gives the ratings for the beam strength tests at each of the four ages; Figure 6, the ratings for core tests at one year; Figure 7, the average uncracked slab length ratings in per cent; Figure 8, the corresponding data in feet, and Figure 9, the average ratings of the various cures, arranged in order of descending value when measured in terms of extent of cracking as compared with the corresponding standard-cure sections.

Beams Under Cure

On reference to Figure 5, it is immediately apparent that, in the great majority of cases, the strength of earth and water cured beams was not attained by the experimental methods. The data also indicate in most cases a falling off in relative strength for the research cure with increase in age. This condition would indicate one of two things: Either that the particular manner of curing the test specimens used in this investigation was at fault, or that the curing method, in itself, was inadequate to retain the moisture. The authors will admit the probability of the first assumption being the true one, and repeat their recommendation that, in future work, strength data on effect of curing be obtained in the laboratory under standardized conditions or that arrangements be made to take specimens from the full-size structure.

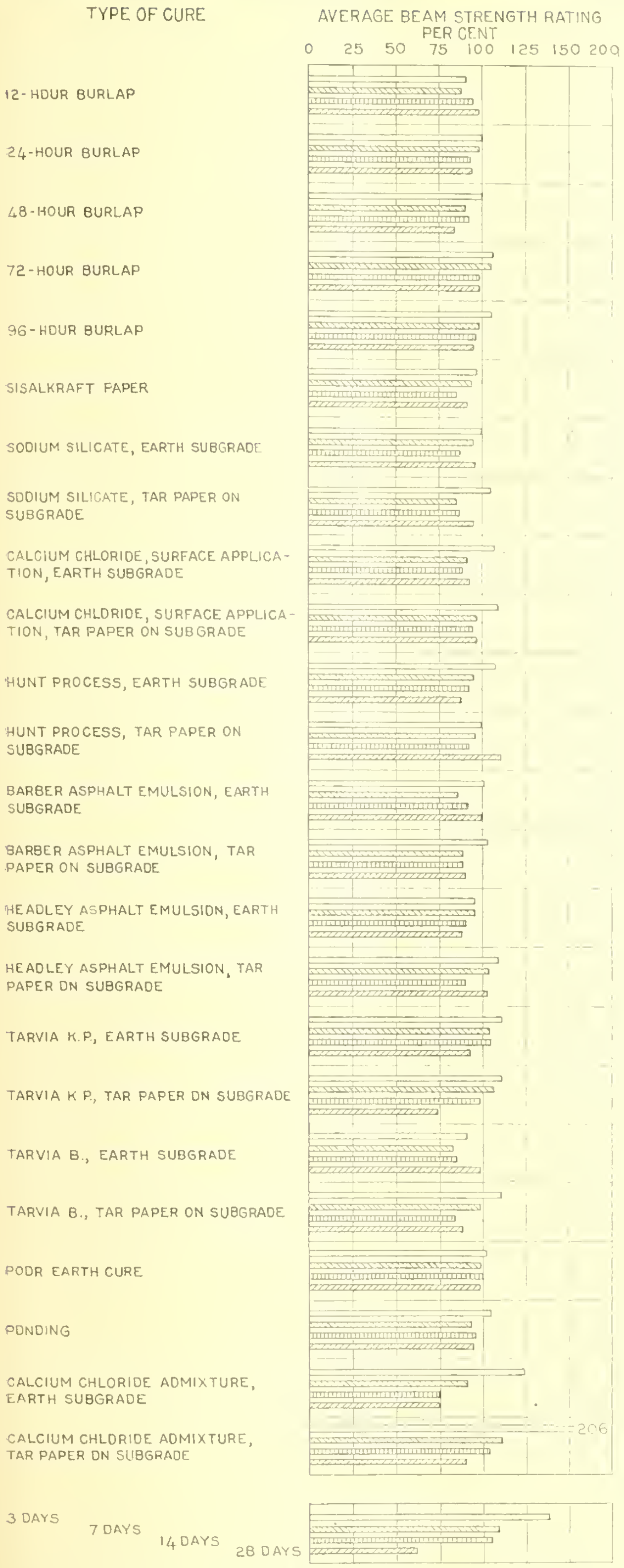


Figure 5.—Relative Strengths of Molded Beams at Various Ages Compared With Similar Specimens Cured by the Standard Wet-Earth Method. Average Beam Strength Ratings are Expressed as Percentages of the Strengths Obtained for Corresponding Standard-Cured Beams

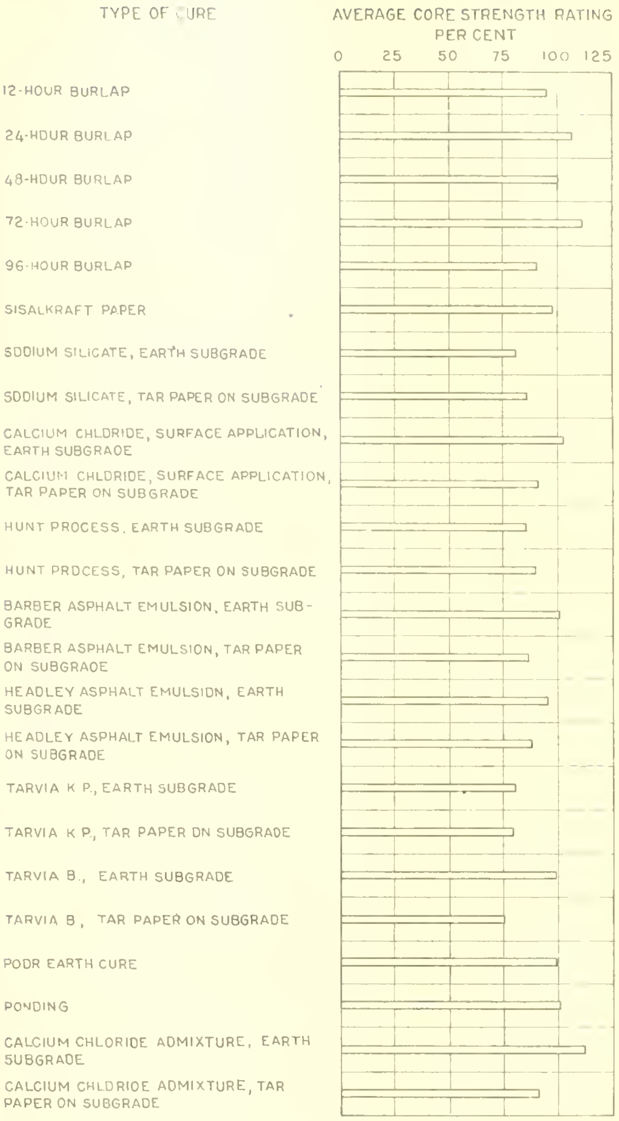


Figure 6.—Relative Strengths of Concrete Cores at One Year Compared With Similar Specimens Drilled From Concrete Cured by the Standard Wet-Earth Method. Average Core Strength Ratings are Expressed as Percentages of the Strengths Obtained for Corresponding Standard-Cured Cores

The average crushing strength ratings shown in Figure 6 are also of interest in that they indicate lower relative strength in the case of sodium silicate and most of the bituminous cures. Burlap cures in general as well as ponding and poor earth cure show results as good as or better than those given by the standard cure. Calcium chloride, both as a surface application and as an admixture, shows relatively high average crushing strength in the case of the sections laid directly on earth subgrade, and relatively low strength in the case of those sections which were laid on tar paper. No explanation is offered for this and the authors are inclined to believe that it is not significant. Attention is also called to the fact that of the eight cases where earth subgrade sections may be compared to similar sections laid on tar paper, six sections show lower relative strength when laid on tar paper. This difference seems to be quite well marked in the case of curcrete, Tarvia B and the calcium chloride sections. A reversal, however, is indicated for Hunt Process and sodium silicate.

Drilling Cores for Compression Tests

Figures 7 and 8 give the average results in regard to cracking. The basis of comparison in Figure 7 is rating in terms of corresponding earth cure, whereas in Figure 8 the comparison is on the basis of average lengths in feet. Figure 7 is of interest in showing that in nearly all cases the extent of cracking is greater on the experimental side of the pavement. Except in the case of the bituminous cures, the difference is not great. In fact there was a general tendency for nearly the same number of cracks to occur on both sides of the longitudinal joint. The graphical record of the crack survey, which is omitted from this article for lack of space, shows that the greater number of these cracks are continuous across both sides of the pavement, a few additional cracks occurring on one or both sides. The fact that so many cracks were carried across the longitudinal joint indicates that there was considerable restraint along this joint, in spite of the precautions which were taken to prevent it. Because of this condition, Figure 8, giving the average uncracked slab length, is the better criterion for judging the relative efficiency of the different research cures. In the comparison of a given research cure with the standard cure, Figure 7, giving the slab length rating, is significant in spite of the existence of restraint across the joint, for the presence of additional cracks on one side of the pavement or the other indicates inferior curing. It is to be recognized, of course, that the variable weather conditions under which the numerous test sections were laid made precise comparisons impossible. However, the averaging of data for sections laid at various times during the construction season tends to iron out the effect of these varying conditions, permitting general deductions to be drawn.

Judged by either Figure 7 or Figure 8, the bituminous cures show unfavorable results in comparison with the standard earth and water cure and the other research cures. The effect of restraint across the joint is clearly indicated by the fact that the standard-cured sections laid opposite to those subjected to bituminous cures show average slab lengths much less than the standard-cured sections laid opposite to the other types of cure. It will be observed that, among the research-cured sections, those cured with burlap, ponding, and poor earth show the greatest uncracked section lengths (about 55 feet) with calcium chloride, Sisalkraft paper, sodium silicate, and the several bituminous cures following in the order named. It will also be noted that very little cracking occurred between December 3, 1929, and May 26, 1930.

SIGNIFICANT DATA SUMMARIZED FOR VARIOUS TYPES OF CURING

The object of a concrete pavement curing agent is not only to supply moisture to the concrete but also to prevent or minimize the internal stresses which may develop during the early hardening period. These internal stresses may be caused by loss of moisture or excessive temperature change or both, and will of course develop transverse cracking in proportion to their magnitude. It should be possible therefore to measure the extent to which the various treatments function in this regard by noting the relative amount of cracking which has developed, especially in those sections subjected to the most severe conditions of temperature change and lack of rainfall. In drawing the conclusion to this report, major consideration has therefore been given to the behavior of those sections upon which the least rain fell during the normal 10-day curing period. The report will be summarized with a brief statement regarding the behavior of that section for each curing method which, from the above standpoint, gave the method its severest test. Certain curing methods are not represented in this summary. Ponding, for example, is not affected by rainfall. In the case of calcium chloride admixture considerable rain fell on all the sections, which were, moreover, of insufficient length to afford a very significant comparison with other methods. The sections cured with Sisalkraft paper were also subjected to considerable rainfall. As regards Tarvia K. P., the section laid September 13, on which no rain fell, was only 259 feet in length. For these reasons it was thought best to omit these types of cure from consideration in the final summary.

Bituminous-Cured and Standard-Cured Sections After Formation of Cracks. Note Crack Spacing on Either Side of Longitudinal Joint

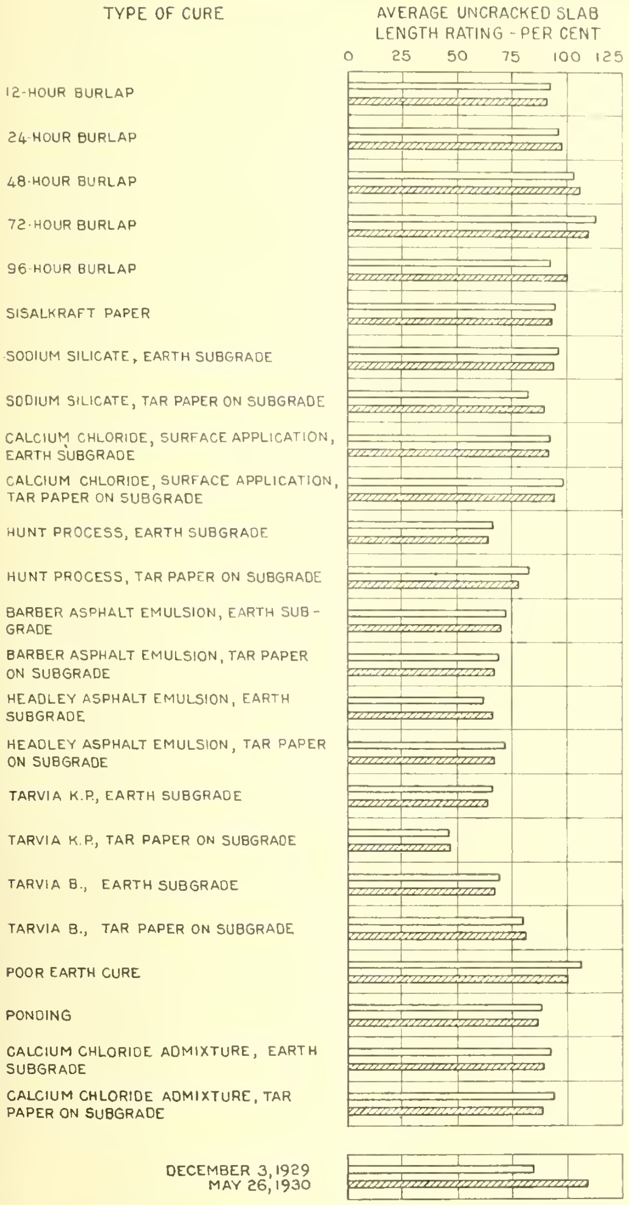


Figure 7.—Relative Average Length of Uncracked Slab Compared With Corresponding Sections Cured by the Standard Wet-Earth Method. Average Uncracked Slab Length Ratings are Expressed as Percentages of the Average Lengths for the Corresponding Standard-Cured Sections

1. *Burlap applied for 24 hours.*—Section laid August 24, 1,022 feet in length. Average temperature during ten days, maximum 85°, minimum 78°. One and one-half inch rain on the seventh day, one inch rain on the 8th day.

Average slab length rating, per cent	119
Average slab length, feet	64
Core strength rating at one year, per cent	102

2. *Burlap applied for 96 hours.*—Section laid August 30, 867 feet in length. Average temperature during ten days, maximum 81°, minimum 61°. One and

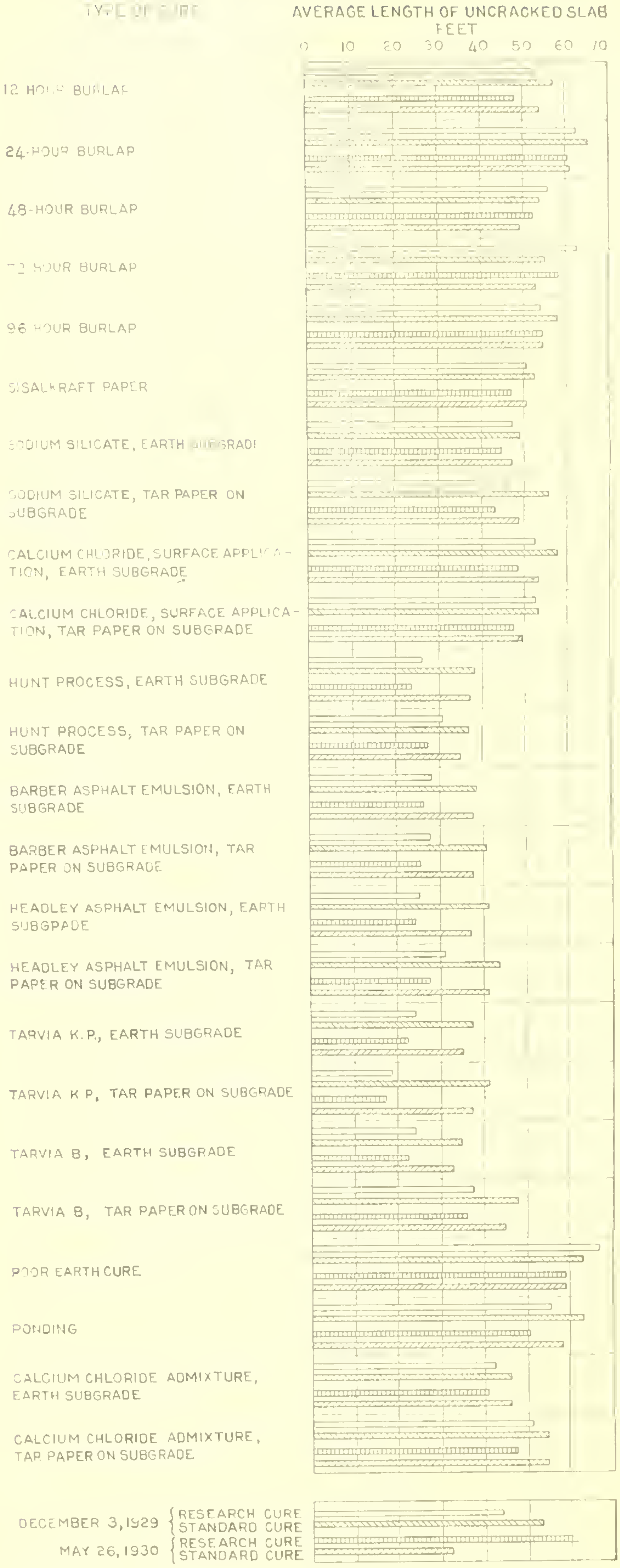


Figure 8.—Results of Crack Survey. Average Length of Uncracked Slab for Each Type of Cure Compared With That for the Corresponding Standard-Cured Section

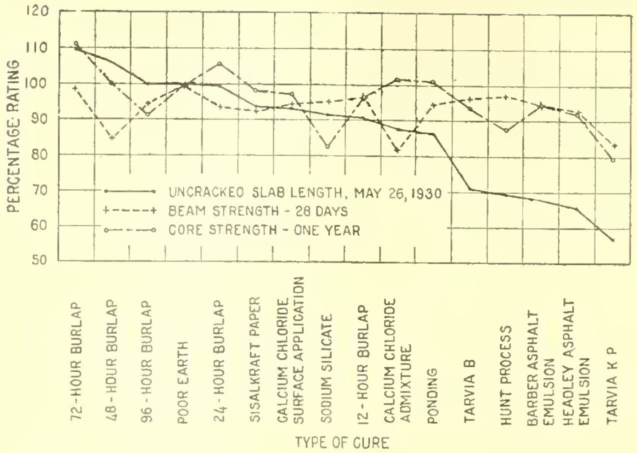


Figure 9.—Comparison of Average Ratings in Respect to Uncracked Slab Length, Beam Strength at 28 Days, and Core Strength at One Year, for Various Curing Methods. The Curing Methods are Arranged in Descending Order of Average Uncracked Slab Length Rating

one-half inches rain on first day and one inch rain on second day. No further rain until October 2. All rain fell before burlap was removed.

Average slab length rating, per cent	100
Average slab length, feet	54
Core strength rating at one year, per cent	92

3. *Sodium Silicate.*—Section laid September 6, 1,087 feet in length. Average temperature during ten days, maximum 89°, minimum 68°. No rain on this section for 26 days.

Average slab length rating, per cent	85
Average slab length, feet	40
Core strength rating at one year, per cent	85

4. *Calcium chloride, surface application.*—Section laid September 4, 588 feet in length. Average temperature during ten days, maximum 87°, minimum 66°. No rain on this section for 28 days.

Average slab length rating, per cent	100
Average slab length, feet	39
Core strength rating at one year, per cent	96

5. *Hunt Process.*—Section laid August 1, 2, 1,459 feet in length. Average temperature during ten days, maximum 92°, minimum 78°. One-eighth inch rain on August 2 (from 6 to 7 p. m.).

Average slab length rating, per cent	86
Average slab length, feet	29
Core strength rating at one year, per cent	73

6. *Barber asphalt emulsion* (*Curcrete*).—Section laid September 11, 1,097 feet in length. Average temperature during ten days, maximum 86°, minimum 64°. No rain fell on this section for 21 days.

Average slab length rating, per cent	70
Average slab length, feet	20
Core strength rating at one year, per cent	119

7. *Headley asphalt emulsion.*—Section laid September 21, 909 feet in length. Average temperature for 10-day period, maximum 79°, minimum 52°. No rain fell on this section for 11 days.

Average slab length rating, per cent	92
Average slab length, feet	35
Core strength rating at one year, per cent	99

8. *Tarvia B.*—Section laid September 17, total length 1,159 feet. Average temperature during ten days, maximum 77°, minimum 52°. No rain fell on these sections for 15 days.

Average slab length rating, per cent	[2] 71
Average slab length, feet	[2] 28
Core strength rating at one year, per cent	[2] 86

9. *Poor earth cure.*—Section laid September 18, 1,121 feet in length. Average temperature during ten days, maximum 78°, minimum 52°. No rain fell on this section in 14 days.

Average slab length rating, per cent	94
Average slab length, feet	70
Core strength rating at one year, per cent	101

CONCLUDING STATEMENT

It is believed that the results of this investigation justify the following comments relative to the merits of the various curing methods as compared with the standard method of curing with earth and water, under the conditions pervailing during the progress of these tests.

1. *Burlap curing.*—Results indicate that the method of cure involving the application of wet burlap for periods varying from 24 to 96 hours without further curing compares quite favorably with the standard method. No final conclusions can be drawn until additional tests under more severe weather conditions are made.

2. *Sisalkraft paper.*—The rather meager results indicate that the application of Sisalkraft paper for periods of 24 to 48 hours without further curing is a reasonably satisfactory method of cure as compared with the standard method.

3. *Poor earth cure.*—Indications are that it is not necessary to keep the earth saturated with water for the entire 10-day curing period.

4. *Sodium silicate.*—Indications are that this method of cure gives results somewhat less satisfactory than those given by the standard method.

5. *Calcium chloride, surface application.*—This method appears to give results very nearly as satisfactory as the standard method.

6. *Calcium chloride admixture.*—The authors feel that the sections on which the admixture was used are too short to warrant any conclusion as to the merit of this method.

7. *Bituminous materials.*—Indications are definite that curing with bituminous materials, as used in these test, is unsatisfactory, because of the marked evidence of increased transverse cracking.

8. *Tar paper on subgrade.*—Results indicate that nothing is gained by the use of tar paper on the subgrade.

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[2] Average of section laid on earth and section laid on tar paper.

RECENT PUBLICATIONS OF THE AMERICAN SOCIETY FOR TESTING MATERIALS

THE American Society for Testing Materials has recently issued its Book of A. S. T. M. Standards, a triennial publication containing all specifications, methods of testing, and definitions fully approved and adopted, and the 1930 Book of A. S. T. M. Tentative Standards, an annual publication, containing those specifications, methods of testing, etc., having a tentative status. The society has also published its annual Index to A. S. T. M. Standards and Tentative Standards.

BOOK OF A. S. T. M. STANDARDS

The Book of A. S. T. M. Standards is published in two parts, Part I on metals (1,000 pages) containing the standards relating to metallic materials and Part II on nonmetals (1,214 pages) containing standards covering nonmetallic materials.

Part I.—Of the 179 standards on metals published in Part I, 105 cover the ferrous metals, steel, cast iron, wrought iron, alloy steel, and ferro-alloys, and 67 relate to the nonferrous metals, nickel, copper, aluminum, etc., and many alloys, while 7 are of general interest.

The standards in Part I, assembled in a sequence determined by the specific materials or products to which they apply cover steel rails and accessories; wheels and tires; structural and boiler steels; steel for welding, concrete reinforcement steel; bar steels; spring steel and springs; steel castings, chain, forgings, and axles; steel tubes and pipe; tool steel, steel for high-temperature service; zinc-coated wire and wire products; wrought-iron bars, castings, plates, and pipe; pig iron; cast-iron and finished castings; malleable castings and ferro-alloys. The specifications in the nonferrous group cover ingot copper, zinc, lead, nickel, aluminum and aluminum alloys, copper wire and cable, brasses and bronzes, solder metal, white-metal bearing alloys, copper and brass plates and tubes. Methods are also included for Brinell hardness tests, metallographic testing, preparation of micrographs for metals and alloys, and a recommended practice for radiographic testing of metal castings. Definitions of terms relating to wrought iron, to methods of testing, to metallography and to specific gravity are also included. New standard specifications have been adopted this year for steel tie-plates, iron and steel chain, gray-iron castings for valves, seamless copper tubing, and bronze and hard-drawn copper trolley wire, as well as several specifications for galvanized wire and wire products and specifications for zinc (hot galvanized) coatings on structural steel shapes and plates. Included in addition to these specifications are new standard methods of sampling rolled and forged steel products for check analysis and test methods for galvanized wire and wire products and a test for change of resistance with temperature of electrical heating materials. Standard specifications for open-hearth steel rails, concrete reinforcement bars, steel pipe and boiler tubes, hot-rolled bar steels and cold-finished shafting, malleable castings, and wrought-iron bars, plates, and pipe that were revised during the year have also been included.

Part II.—The 251 standards in Part II cover the following miscellaneous groups of nonmetallic materials and products: Cement, lime, gypsum, concrete and concrete aggregates; brick and refractories; pipe and drain tile; hollow building tile; paints, pigments, shellacs, and varnishes; petroleum products and lubricants; bituminous and nonbituminous road materials; coal and coke; timber and timber preservatives, waterproofing and roofing materials; insulating materials and rubber products; textile materials; and thermometers for general use. Included in this volume are new standards adopted this year comprising specifications for paving and building brick; sand-lime brick; wall, floor, and partition hollow clay tile; Keene's cement and gypsum plasters; gravel for bituminous concrete; several specifications for tar cements for road application; tolerances for cord-tire fabrics and other fabrics; methods of testing concrete aggregates, gypsum, and gypsum products; analysis for color of paints; tests for sulphur in gasoline; melting point of petrolatum; test for autogenous ignition temperatures of petroleum products; mechanical analysis of coal; and test methods for electrical porcelain. The specifications for Portland cement, revised to include higher tensile strength requirements, and the methods of testing cement, changed by the inclusion of tolerances on weights and dimensions of apparatus, are also included. Of particular interest are the revised standard specifications for structural wood, joists and planks, beams and stringers, and posts and timbers.

1930 BOOK OF A. S. T. M. TENTATIVE STANDARDS

The Book of Tentative Standards (864 pages), an annual publication, contains 155 tentative specifications, methods of test, definitions of terms, and recommend practices, 28 relating to metals and 127 to nonmetallic materials and products. The tentative standards are issued for one or more years with a view to obtaining criticisms. Although in the trial stage of A. S. T. M. procedure, they represent the latest thoughts of the committees on the subjects covered and are therefore being applied in the various industries. Many of the tentative standards are frequently used in conjunction with the A. S. T. M. standards.

Included in this volume are 21 new tentative standards developed this year. In the metals group are new specifications for austenitic manganese-steel castings, open-hearth iron plates; aluminum alloy (duralumin) sheet; aluminum-manganese alloy sheet and magnesium-base alloy castings; aluminum-base alloy castings and ingots and copper-base alloys in ingot form; and a method of test for thermoelectric power. A specification for high early-strength Portland cement is an important contribution to the construction field, as are also new specifications for reinforced-concrete pipe and reinforced-concrete culvert pipe, and specifications for gypsum sheathing board, timber piles, glazier's putty, test methods for natural building stone, and a flexure test for concrete. Of interest in the automotive field are the test for dilution of crankcase oil and a test for vapor pressure of natural gasoline. A test method is also included for comparing the thermal conductivities of solid electrical insulating materials. New tentative specifications have been developed for several lacquer solvents and diluents, namely, amyl acetate, amyl alcohol, butyl propionate, and ethyl lactate. Revisions were also made in 25 of the tentative standards developed in former years in order to keep them abreast of current practice.

INDEX TO A. S. T. M. STANDARDS AND TENTATIVE STANDARDS

An index (110 pages) of all A. S. T. M. standards and tentative standards has been prepared and is being distributed without charge. The index is a compilation, under appropriate key-word subjects covered by the standards, of titles of all standards, together with volume references to the publications in which they appear, namely, Parts I and II of the 1930 Book of A. S. T. M. Standards and 1930 Book of A. S. T. M. Tentative Standards and the Annual Proceedings. The index is designed to be of service to those familiar with the society's standards in locating specifications, methods of test, or definitions in the bound publications in which they appear, as well as to those interested in ascertaining if the society has issued any standards on a specific subject.

INDEXES TO VOLUMES NOS. 8 AND 9 OF PUBLIC ROADS AVAILABLE

Indexes to volumes 8 and 9 of Public Roads have recently been printed and are available for distribution upon request. The indexes to volumes 6 and 7 were issued some time ago, and a supply of these indexes is still available for distribution. It is planned to issue indexes to volumes 10 and 11 during this year and thereafter to issue an index annually.

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ROAD PUBLICATIONS OF BUREAU OF PUBLIC ROADS

Applicants are urgently requested to ask only for those publications in which they are particularly interested. The Department can not undertake to supply complete sets nor to send free more than one copy of any publication to any one person. The editions of some of the publications are necessarily limited, and when the Department's free supply is exhausted and no funds are available for procuring additional copies, applicants are referred to the Superintendent of Documents, Government Printing Office, this city, who has them for sale at a nominal price, under the law of January 12, 1895. Those publications in this list, the Department supply of which is exhausted, can only be secured by purchase from the Superintendent of Documents, who is not authorized to furnish publications free.

ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

DEPARTMENT BULLETINS

No. *136D. Highway Bonds. 20c.
*314D. Methods for the Examination of Bituminous Road Materials. 10c.
*347D. Methods for the Determination of the Physical Properties of Road-Building Rock. 10c.
*370D The Results of Physical Tests of Road-Building Rock. 15c.
*463D. Earth, Sand-Clay, and Gravel Roads. 15c.
*532D. The Expansion and Contraction of Concrete and Concrete Roads. 10c.
*583D. Reports on Experimental Convict Road Camp, Fulton County, Ga. 25c.
*660D. Highway Cost Keeping. 10c.
*670D. The Results of Physical Tests of Road-Building Rock in 1916 and 1917.
*691D. Typical Specifications for Bituminous Road Materials. 10c.
*724D. Drainage Methods and Foundations for County Roads. 20c.
1216D Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1279D Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

No 55T. Highway Bridge Surveys.

MISCELLANEOUS CIRCULARS

No. 62M. Standards Governing Plans, Specifications, Contract Forms, and Estimates for Federal-Aid Highway Projects.
*93M. Direct Production Costs of Broken Stone. 25c.
109M. Federal Legislation and Regulations Relating to the Improvement of Federal-Aid Roads and National-Forest Roads and Trails, Flood Relief, and Miscellaneous Matters.

MISCELLANEOUS PUBLICATIONS

No. 76MP. The Results of Physical Tests of Road-Building Rock.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *914Y. Highways and Highway Transportation. 25c.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio.

Report of a Survey of Transportation on the State Highways of Vermont.

Report of a Survey of Transportation on the State Highways of New Hampshire.

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio.

Report of a Survey of Transportation on the State Highways of Pennsylvania.

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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* Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL-AID ROAD CONSTRUCTION

AS OF

DECEMBER 31,1930

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	2,178.0	$ 3,496,835.94	$ 1,674,991.27	130.2	26.8	157.0	$ 1,458,484.08	$ 722,810.35	70.9	4.2	75.1	$ 5,158,890.72	Alabama
Arizona	892.1	3,554,582.34	2,793,740.90	135.6	142.3	277.9	413,986.52	288,574.87	7.9	10.0	17.9	2,237,201.64	Arizona
Arkansas	1,806.9	6,017,891.84	2,805,258.89	179.2	46.7	225.9	1,222,178.99	605,570.13	23.7	13.5	37.2	2,772,812.42	Arkansas
California	1,945.3	7,353,774.31	3,105,385.74	166.6	25.1	191.7	1,838,958.45	878,586.66	26.2	5.9	32.1	4,075,099.79	California
Colorado	1,284.4	4,135,823.75	2,195,311.34	137.8	57.9	195.8	871,897.88	489,545.35	38.7	32.5	71.2	3,736,815.25	Colorado
Connecticut	252.9	2,190,070.59	959,644.81	12.9		12.9	455,213.95	84,060.00	5.6		5.6	1,252,645.45	Connecticut
Delaware	318.9	106,532.85	53,315.42	4.4		4.4	320,693.10	153,372.65	8.5		8.5	495,885.89	Delaware
Florida	533.8	4,537,263.81	2,206,061.92	76.0	5.5	81.5	754,310.87	348,842.51	27.4		27.4	2,588,374.50	Florida
Georgia	2,662.4	6,258,848.20	2,884,108.86	156.3	123.8	280.1	2,992,194.25	1,391,594.41	84.2	37.9	122.1	4,197,351.83	Georgia
Idaho	1,292.1	2,093,187.46	1,277,605.17	143.5	11.8	155.2	712,959.91	428,013.57	58.4	8.2	66.6	1,630,831.81	Idaho
Illinois	2,167.3	20,382,857.78	9,251,863.43	576.2	68.6	644.8	1,357,755.66	645,586.15	47.7		47.7	7,133,258.00	Illinois
Indiana	1,569.1	2,332,075.25	1,133,362.55	75.3		75.3	4,641,094.20	1,593,935.86	98.8		98.8	4,003,555.80	Indiana
Iowa	3,229.0	105,290.11	1,892.77		4.0	4.0	2,810,112.18	1,151,015.00	41.6	47.8	89.4	2,194,055.62	Iowa
Kansas	2,973.8	4,785,313.73	2,247,984.50	258.4	31.6	290.0	2,878,742.08	1,306,112.46	169.4	52.9	222.3	3,069,397.74	Kansas
Kentucky	1,519.3	5,870,327.62	2,346,264.11	166.2	85.0	251.2	2,468,783.53	555,481.88	46.9	28.3	75.2	1,480,640.71	Kentucky
Louisiana	1,409.7	4,344,555.40	2,104,665.85	166.8	18.1	175.9	2,046,277.88	925,702.88	60.1	12.7	72.8	1,888,529.98	Louisiana
Maine	580.4	2,518,595.24	848,864.82	54.5		54.5	542,428.70	271,214.35	.4		.4	1,918,144.20	Maine
Maryland	684.7	1,018,034.12	473,123.38	9.0	13.2	22.2	158,535.70	41,957.10		2.4	2.4	999,144.32	Maryland
Massachusetts	711.8	6,762,321.06	1,802,841.34	52.8		52.8	1,894,170.88	378,667.93	24.5		24.5	2,468,042.29	Massachusetts
Michigan	1,786.1	7,941,942.34	3,299,486.53	202.3	22.9	225.2	1,149,845.53	505,940.00	41.8	1.5	43.3	4,185,778.95	Michigan
Minnesota	4,314.0	2,141,865.83	790,564.13	23.1	92.9	116.0	5,172,293.18	1,895,745.64	57.3	156.4	213.7	880,822.68	Minnesota
Mississippi	1,830.0	779,218.74	385,918.51	35.7	17.8	53.5	518,222.51	259,111.25		29.8	29.8	5,371,325.81	Mississippi
Missouri	2,588.5	8,630,891.81	3,085,330.09	147.3	54.1	201.4	1,814,392.70	758,271.18	66.3		66.3	2,778,159.75	Missouri
Montana	1,892.3	7,343,731.48	4,144,714.08	573.6	40.9	614.5	844,387.72	458,157.47	98.0	11.5	109.5	4,806,823.57	Montana
Nebraska	3,818.3	5,067,558.32	2,427,549.44	269.5	96.5	366.0	1,470,173.25	532,227.18	51.6	20.8	72.4	3,696,667.01	Nebraska
Nevada	1,278.0	1,005,734.41	871,720.23	52.5	63.4	115.9	10,142.81	9,011.88		.8	.8	1,881,583.08	Nevada
New Hampshire	382.3	1,039,256.81	377,839.66	17.7	1.8	18.3						572,280.18	New Hampshire
New Jersey	551.1	3,295,951.94	825,666.32	23.4		23.4						2,548,366.75	New Jersey
New Mexico	1,957.5	4,149,839.98	2,753,339.06	187.1	51.9	239.0	350,278.43	300,409.23	14.3		14.3	2,358,585.03	New Mexico
New York	2,665.2	18,553,810.33	3,512,755.00	243.8		243.8	4,892,246.10	1,179,345.50	79.1		79.1	11,774,405.54	New York
North Carolina	1,882.7	4,013,858.02	1,917,847.56	128.3	33.2	161.5	1,090,869.13	520,260.11	61.4	14.7	76.1	4,024,001.81	North Carolina
North Dakota	4,593.2	1,588,804.02	843,299.08	219.8	133.2	353.0	1,181,379.47	590,307.53	180.2	126.6	306.8	2,601,532.32	North Dakota
Ohio	2,504.4	13,749,210.96	4,257,180.04	194.6	18.5	213.1	630,333.50	179,798.24	2.6	1.2	3.8	4,509,133.68	Ohio
Oklahoma	1,943.9	5,182,867.33	2,463,515.75	138.5	87.8	226.3	1,166,751.51	641,882.51	85.6	7.3	92.9	2,278,994.68	Oklahoma
Oregon	1,229.2	5,472,332.47	3,218,383.67	221.2	75.3	296.5	894,046.45	509,999.04	32.9	3.4	36.3	1,583,475.38	Oregon
Pennsylvania	2,655.6	7,520,500.18	2,593,140.26	48.8		48.8	64,282.12	24,836.06	.5		.5	5,148,520.04	Pennsylvania
Rhode Island	209.5	1,720,027.14	664,901.84	25.4		25.4						626,545.33	Rhode Island
South Carolina	1,850.7	5,957,224.17	2,412,013.31	108.4	122.7	231.1	779,057.73	387,468.70	.2	27.1	27.3	1,220,875.55	South Carolina
South Dakota	3,795.8	2,829,464.55	1,564,585.70	276.4	76.2	352.6	635,974.55	350,335.92	90.7	18.7	110.4	2,284,864.57	South Dakota
Tennessee	1,329.2	4,331,480.98	1,948,251.19	162.7	41.4	204.1	806,731.68	403,355.83	19.7		19.7	3,306,324.89	Tennessee
Texas	6,954.2	13,340,855.59	5,631,065.92	540.8	158.4	699.3	2,568,987.69	994,464.59	146.6	47.5	194.1	9,840,581.79	Texas
Utah	1,036.6	592,233.93	511,813.03	66.7	20.0	86.7	807,205.22	582,203.41	32.0	55.9	87.9	1,395,711.68	Utah
Vermont	299.7	384,278.14	80,949.10	5.5		5.5						613,889.05	Vermont
Virginia	1,613.8	3,758,942.60	1,803,700.57	151.2	9.4	160.6	585,192.00	248,550.81	14.4	5.1	19.5	2,213,225.90	Virginia
Washington	983.5	2,963,271.03	1,282,600.00	55.1	36.3	91.4	1,540,845.73	737,000.00	59.5		59.5	2,237,160.58	Washington
West Virginia	724.3	4,658,999.02	1,695,549.73	114.2	34.1	148.3	828,840.20	347,496.27	8.5		8.5	1,085,032.06	West Virginia
Wisconsin	2,467.0	3,870,493.22	1,692,500.00	101.1	17.6	118.7	179,803.32	85,000.00	8.8		8.8	2,577,770.40	Wisconsin
Wyoming	1,755.6	2,635,627.51	1,755,255.52	179.4	91.3	270.7	553,044.41	359,603.25	5.5	82.5	88.0	1,166,957.70	Wyoming
Hawaii	41.2	882,655.31	374,008.53	21.7		21.7	197,730.75	98,865.37	9.0		9.0	1,852,759.58	Hawaii
TOTALS	88,945.3	233,397,04[illegible].62	99,572,832.92	7,029.9	2,058.7	9,088.6	50,393,535.17	25,530,501.36	2,007.1	868.3	2,875.4	144,725,775.57	TOTALS

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[1] The term stage construction refers to additional work done on projects previously improved with Federal aid In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE 1931

PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE
BUREAU OF PUBLIC ROADS

VOL. 11, NO. 12 ▽ FEBRUARY, 1931

TWO-BATCH TRUCK WAITING FOR THE SKIP TO COME DOWN

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PUBLIC ROADS

A JOURNAL OF HIGHWAY RESEARCH

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

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The reports of research published in this magazine are necessarily qualified by the conditions of the tests from which the data are obtained. Whenever it is deemed possible to do so, generalizations are drawn from the results of the tests; and, unless this is done, the conclusions formulated must be considered as specifically pertinent only to the described conditions

VOL. 11, NO. 12 **FEBRUARY, 1931** **G. P. St. CLAIR, Editor**

TABLE OF CONTENTS

Page

North Carolina County Road and Finance Survey 237

Truck Operation and Production in Concrete Paving Work 247

High-Speed Production on Asphalt Concrete Paving Work 258

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NORTH CAROLINA COUNTY ROAD AND FINANCE SURVEY

Report of a Cooperative Investigation by the North Carolina State Highway Commission, the North Carolina State Tax Commission, and the United States Bureau of Public Roads.[1]

UNDER the general direction of Gov. O. Max Gardner, a cooperative survey of county road mileage and financial operations of the counties of North Carolina was undertaken July 16, 1930, by the State Highway Commission of North Carolina, the State Tax Commission of North Carolina, and the Bureau of Public Roads of the United States Department of Agriculture.

> The cooperative survey of county road mileage and finance which was conducted in North Carolina during the latter half of 1930 revealed the existence of numerous defects in organization and administration. It is believed that many of the conditions found in North Carolina are characteristic of other counties throughout the United States. For this reason the text of the report on this survey is presented here, in the hope that it will prove of value to those interested in promoting efficiency and economy in the construction, maintenance, and financing of county roads. A number of tables giving data for the individual counties, which were included in the report as submitted to the Governor of North Carolina, are omitted here. With this exception, all the material which appeared in the original report is included in this article.
>
> The first part of the report discusses the manner in which the survey was conducted and the data which were obtained from the various counties. In the second section a résumé is made of the facts disclosed by the investigation. The third section consists of recommendations for the improvement of conditions in the counties of North Carolina.

The purposes of the survey were to ascertain for each county the mileage and location of county and township roads, and to determine the amounts and sources of road revenue, the purpose for which expenditures of road revenue were made, the amount of bonded indebtedness outstanding for road purposes, rates of taxation levied for road purposes, and other information which would be of use in a general examination of the local road finance and management in North Carolina.

Because of the limited time available for this investigation, certain features of the survey were made on a reconnaissance basis. Thus, road mileage was measured with speedometers, road locations were determined from existing maps, and where tax settlements had not been made by the sheriffs a reasonable estimate of probable collections made by county officials was accepted.

The actual work of collecting road-mileage data and the preparation of the maps was undertaken by the State highway commission, operating through the nine divisions of the commission. The collection of data with regard to highway income, expenditures, bonded indebtedness, tax rates, etc., was undertaken by the State tax commission and the United States Bureau of Public Roads, operating through field parties which visited each county seat and obtained data from the county records. The task of analyzing county records was greatly simplified by the fact that the legislature of 1927 created the county advisory commission, which had set up for nearly all counties a standard accounting procedure which was accepted and operated by the counties in a very creditable manner.

North Carolina, with a total area of 52,426 square miles, has a land area of 48,744 square miles, which has been used as a basis for comparison with road mileage. This area is divided into 100 counties, with an average area of 487.4 square miles.

ORGANIZATION

The financial affairs of each county are administered by an elective board of county commissioners consisting of three, five, or seven members. The total number of commissioners for all counties during 1930 was 407.

The county board fixes tax levies, sets up and approves budgets, and, in general, under the statutes, exercises the powers of a legislative and administrative body. In 24 counties the board also serves as a county highway board having jurisdiction over the expenditure of highway funds. In 44 counties the highway board consists of an independent membership composed of from one to seven members. These boards supervise expenditure for road purposes only and do not have power to fix tax rates. In 26 counties some form of township road organization exists, composed of from one to three members with varying powers over road matters. In several counties the township boards levy taxes, collect and expend the proceeds without county supervision. In others, the levy is fixed and collected by the county and expended by the township.

County engineers appointed by the county boards are found in 25 counties. In 18 of these counties, road superintendents are also employed. In 49 counties actual supervision of road work is vested in a county superintendent of roads, who is employed by the board and receives instructions from that body. In the remaining counties the members of the board of commissioners also serve as heads of the road forces in the townships comprising their respective districts.

The lack of uniformity in local road organizations is due primarily to special legislation enacted by the legislature permitting deviations from a standard organization, or to the enactment of special laws which set up new forms of management and organization.

FINANCES

The local road program of the fiscal year ending June 30, 1930, was financed almost entirely from local tax levies. Although approximately $3,000,000 was made available to the counties from State funds, only 15 of the 100 counties elected to spend their share on roads, the remainder applying their contribution to the debt service fund used to pay interest and principal on outstanding county obligations.

The legislature of 1929 made available to the counties the proceeds of the additional 1-cent tax on gasoline. This sum, with $500,000 from the regular State highway fund, comprised the $3,000,000 fund available to counties for expenditure for road purposes under the supervision of the State highway commission or to be expended for retirement of outstanding indebtedness under the supervision of the State debt commission.

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[1] This survey was executed during the period July 16 to Dec. 15, 1930, by the personnel of the organizations cooperating. The work was carried on under the direction of R. A. Doughton, chairman of the State highway commission, and A. J. Maxwell, chairman of the State tax commission. The following employees contributed full-time service during this investigation and assisted in the preparation of the final report and maps: Messrs. T. M. Denson and C. M. Sawyer, of the State highway commission; Fred W. Morrison and Prof. Mark C. Leager, of the State tax commission; and George G. Clark, A. E. McClure, and W. A. Blanchette, of the U. S. Bureau of Public Roads.

Table 1, compiled from the records of all counties for 1930, shows the sums made available for county and township road purposes.

TABLE 1.—*Total funds available to all counties of North Carolina for road purposes during the fiscal year 1930*

1. Sale of road and bridge bonds		$100, 059. 36
2. Sale of notes [1]		338, 023. 60
3. Appropriations or transfers from other funds		124, 596. 44
4. Tax collections:		
Current year	$5, 461, 436. 63	
Previous years	276, 361. 25	
		5, 737, 797. 88
5. State funds contributed		283, 165. 14
6. Miscellaneous income		480, 834. 30
Total new funds for year		7, 064, 476. 72
Balance brought forward from preceding year		2, 705, 567. 69
		9, 770, 044. 41

From these figures it will be seen that support of local roads is dependent largely on local tax levies. These levies vary from 7 cents per $100 in Buncombe County to 75 cents per $100 in Ashe County, rates in general varying inversely with the wealth and population of the counties. The total road taxes collected during 1930, current and delinquent, represented about 19.3 cents per $100 for the State as a whole. The average tax rate levied in the 10 wealthiest counties for 1930 was 14.7 cents on the $100, with the highest rate, 19¾ cents, in the tenth county. The other side of the picture is shown in the 10 poorest counties, with levies averaging 21.2 cents and a range of from 7 to 65 cents on the $100.

Of the 26 counties levying township taxes, which amounted in 1929 to a total of $951,667, total levies in 14 counties amounted to only $94,826, or an average of less than $7,000 per county. In the remaining 12 counties levying $856,841, the average was $71,400 per county, with three exceeding $100,000 per county.

The fiscal year for counties begins July 1 of each year. Property taxes are not due until November 1 following, with the result that in most counties the road administrator is confronted during the period of most active work with an empty treasury and in many cases with a deficit. To relieve this condition county boards may sell "tax-anticipation warrants" in amounts not to exceed 80 per cent of the taxes levied. The restriction on the amount of notes sold was placed by the legislature of 1927 and was intended to correct abuses of note selling, which in many instances in previous years were not repaid but were converted into long-term bonds.

BOND ISSUES

The total amount of county and township road bonds outstanding on June 30, 1930, as nearly as could be determined from county reports, was $100,066,572. These bonds, in general, were of the sinking-fund type with 20 to 30 year terms, bearing interest at rates ranging from 4½ per cent to 6 per cent per annum. The weighted average rate for all issues was 5.247 per cent. This total includes only those bonds appearing on the records as issued for road and bridge purposes. These bonds covered loans and donations to the State highway commission, costs of construction of county roads and bridges, and deficits in the road funds of the counties. For many years it has been a more or less general practice of county boards of commissioners to cover a deficit in the road fund with notes which were later covered by bonds. Such road and bridge bonds as have matured and have been covered by issues of funding bonds have, in general, lost their identity on the county records as road funds. Such bonds do not appear in the above total. No reliable estimates of the amount of such funding bonds outstanding properly chargeable as road bonds could be obtained.

The information regarding amounts in sinking funds for the retirement of outstanding bonds was somewhat confused, because of the rather general practice of treating all bond issues as a common debt with a common sinking fund for all. Again, in many instances, casual inquiry developed that sinking funds had been borrowed for other departments of the county or invested in real estate loans, which are in some instances in the status of frozen assets. In general, detailed, accurate information in regard to sinking funds for road and bridge bonds was not available during the progress of this survey. On account of the wide powers vested in local officials by statutes and special legislation to issue bonds, and the promptness with which interest charges have been paid, a ready market for such securities has been found. Therefore, many improvements which might have been financed from current revenues by the imposition of an increased road and bridge tax have been met by the issue of notes which have been later converted into bonds.

County board relations with the State highway commission have brought about transfers of county funds to expedite the construction of particular sections of the State highway system. In general this procedure has involved a loan to the State which has been later repaid, or an outright donation to the State. In a few instances sections of State road have been built by the counties and then transferred to the State highway system. County funds in the amount of $16,799,576.40 have been loaned to the State highway commission, of which only $4,770,651.28 is still outstanding. In addition to this sum the counties have donated to the State outright $8,772,869.93 without expectation of repayment. The State highway commission is making repayments as rapidly as funds become available.

Under the terms of recent State legislation new bonds and refunding bonds issued must be of the serial type and tax levies must be provided for the payment of principal and interest. During 1930 principal requirements of bonds maturing, both of sinking fund and serial types, amounted to $2,324,368, while interest requirements amounted to $5,249,407, giving a total of $7,573,775, a sum in excess of the annual county road expenditures for maintenance and almost as much as the costs of construction and maintenance combined.

Deducting from the outstanding bond issues the sums due from the State highway commission, the donations actually made to the commission in cash, and roads built, there still remains outstanding a sum of bonded indebtedness far in excess of the present worth of the highways remaining on the county systems. By far the greater part of the excess represents deficits from previous years; that is, ordinary road and bridge expenditures converted into bonds by way of notes.

Therefore, the present debt service levy, in part, represents a charge that should have been levied during the past decade when taxes were low and deficits were allowed to accumulate.

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[1] Where notes were repaid during 1930, such items do not appear in these totals.

EQUIPMENT

By means of a questionnaire sent out by the North Carolina County Advisory Commission, an estimate of the road equipment owned by the counties and townships was obtained. These replies showed an estimated cost of equipment on hand to be $2,588,000. Of this amount $1,887,600 was located in 47 counties operating convict forces. The remaining $700,400 was located in the 53 counties dependent on hired labor for road upkeep.

It was not possible with the force at hand and the limited time available to make a check and appraisal of the equipment in each county, but interviews with field representatives of the State highway commission and study of county expenditures indicated that road equipment purchased was in many instances of the heavier and more expensive types. This may possibly explain the gasoline, oil, and grease bill in excess of $900,000 or $20 per mile per year for each mile of county road in the State, many miles of which showed no evidence of having benefited from regular maintenance.

CONVICT CAMPS

Forty-seven counties of the State maintain convict camps or stockades for the incarceration of misdemeanants and felons serving less than 10-year terms. The authority for the organization of county prisoners and short-term felons into county camps is found in chapter 24, articles 2 and 9, and chapter 70, article 4, of the consolidated statutes of North Carolina and numerous public-local laws applicable usually to a single county but occasionally to two or more.

The official in charge of a prison camp is generally termed the superintendent; or he may be the county superintendent of roads; or, in other cases, the county road engineer may exercise general supervision over the camp.

Due to the multiplicity of public-local laws, little uniformity as to organization of camps and utilization of inmates on road work exists. In general, able-bodied prisoners are placed in the road gang and utilized at the discretion of the superintendent.

The records of expenditures and road mileage show that few miles of high types of road surface—i. e., bituminous macadam or better—were constructed during the last year. Therefore, convicts were used largely for the maintenance, repair, and reconstruction of earth, sand-clay and topsoil types of road. Statements of camp officials and local and State road officials tended to establish the opinion that the use of prisoners under the supervision of armed guards on work of the latter type did not produce results at a low unit cost. Several reasons were assigned for this condition: First, the cost of guarding small bodies of men at work on maintenance or small repair jobs; second, the inefficiency of the prisoners; third, the difficulty of obtaining superintendents possessing the necessary knowledge of road building and maintenance combined with the ability to develop efficient work from the prisoners, the latter reason having the greatest effect on the results secured. Many superintendents strive to keep the prisoners at work, but in many counties more emphasis is placed on labor performed than on efficiency of the results secured.

Another vexing phase of the convict problem is the large number of short-term men, many of whom are diseased, who pass through the camp for 30 or 60 day terms. The labor of these men does not reimburse the county for court fees, clothing costs, medical attention and subsistence, all of which are paid from the road fund.

No figures were available as to the average number of convict camp prisoners in the State. Messrs. Steiner and Brown, of the University of North Carolina at Chapel Hill, estimated in 1927 that 2,500 were so confined. This estimate seems to be fair and reasonable. The total expenditure on account of convict camps in 1930, including all costs shown on the county books, was $1,289,686.57, or an average annual cost to the taxpayer of $515 per convict, using 2,500 convicts as an average. Wide variations from this figure may be found in individual counties due to differences in accounting procedure and efficiency in managing the camps.

EXPENDITURES

The total road expenditures of all counties and townships during 1930 reached a total of $8,233,280.93, excluding the transactions covering short-term notes which were executed and paid during the year. Following the arrangement of accounts recommended by the county advisory commission, the expenditures are listed in Table 2.

TABLE 2.—*Total road expenditures of all counties in North Caroline during fiscal year 1930*

Item	Amount	Percentage of total
1. General overhead expense	$271, 856. 41	3. 3
2. Convict gangs	1, 289, 686. 57	15. 6
3. Maintenance of roads and bridges	2, 725, 807. 93	33. 1
4. Construction of roads and bridges	1, 342, 611. 63	16. 3
5. Equipment expense	1, 371, 651. 48	16. 7
6. Interest on short-term notes	70, 745. 78	0. 9
7. Principal of short-term notes	94, 903. 74	1. 1
8. Gasoline, oil, and grease	908, 144. 47	11. 1
9. Miscellaneous expenditures (not classified in the records)	157, 872. 92	1. 9
	8, 233, 280. 93	100. 0

These accounts were made up as follows: Under item 1, all expense and salaries for those road officials employed in or about the court house, including fees of county commissioners; under item 2, all salaries and expense incident to the convict camps as shown on county records; under item 3, all materials, salaries, wages, and other items entering into road maintenance; under item 4, all materials, salaries, wages, and contracts entering into road and bridge construction; under item 5, all expense incident to the repair and operation of equipment except gasoline, oil, and grease; under item 6, interest paid on short-term notes, the proceeds of which were placed in the road fund; under item 7, payments of principal of short-term notes carried forward from the previous year; and under item 8, all purchases of gasoline, oil and grease.

The last-mentioned item, comprising so large a part of the total expenditure, is reported as a separate account, because the records were not kept uniformly throughout the State, as in some counties all gasoline was charged to convict camps, in others to equipment expense, and in some instances to construction or to maintenance; and also to bring out the relationship existing between equipment on hand, the cost of repairs and replacements, and the cost of gasoline, oil, and grease used in operation. Equipment on hand was valued at $2,588,000; equipment repairs and operation except gasoline, oil, and grease, $1,371,651.48; and gasoline, oil, and grease, $908,144.47. The combination of the last two items equaling $2,279,795.95 is more than

25 per cent of the total local expenditures for construction and maintenance.

Another classification of accounts recommended by the North Carolina County Advisory Commission groups expenditures as shown in Table 3.

TABLE 3.—*Alternative classification of road expenditures for all counties*

Group	Amount
1. Personal services	$2, 399, 511. 28
2. Supplies and materials	2, 514, 890. 89
3. Other expenses including road contracts	2, 408, 693. 15
4. Purchases of equipment other than replacements	708, 452. 04
5. Expenditures not classified	201, 733. 57
	8, 233, 280. 93

A further analysis of these groups follows.

Under Group 1, the item of wages (labor pay rolls) leads with an expenditure of $1,713,111.83. Guards for camps cost $254,130.52. Salaries of superintendents amounted to $219,651.04, the remainder of the expenditure falling into small items.

Under Group 2, major expenditures are found as follows:

Gasoline, oil, and grease		$908, 144. 47
Food and provisions		321, 319. 38
Timber and lumber		213, 043. 63
Feed and animal supplies		177, 903. 56
Motor-vehicle supplies and parts		92, 585. 60
Hardware and paints		45, 829. 28
Medicine and drugs		19, 761. 49
Wearing apparel		78, 074. 92
Bedding and linen		18, 794. 18
Small tools		26, 762. 73
Tires and tubes		21, 185. 44
Household and janitor supplies		19, 586. 35
Road materials:		
Explosives	$17, 940. 98	
Cement	10, 161. 69	
Sand	8, 556. 34	
Gravel	21, 989. 40	
Brick	72. 24	
Stone	35, 283. 03	
Pipe, tile, and culverts	117, 314. 56	
		211, 318. 24

The total of road materials purchased, as above stated, is very small, but it must be added that it does not include the materials used on construction contracts as these were not segregated in the county records. Other items in this group were small in amount and of little significance. However, unclassified items totaling $300,396.92 were found listed under this group in the accounts of the counties.

Under Group 3 outstanding expenditures were as follows:

Miscellaneous construction	$1, 479, 818. 09
Blacksmithing and shoeing	11, 423. 80
Repairs to buildings, fences, etc	18, 507. 80
Repairs and replacements to equipment	357, 288. 61
Repairs to automotive equipment	124, 797. 07
Interest and discounts	62, 591. 44
Insurance and bonding	41, 129. 36
Miscellaneous (not classified)	241, 361. 49
Freight, drayage, and express	11, 316. 84
Awards and damages	10, 911. 23
Electric light and gas	10, 960. 06

Other small items less than $10,000 each make up the remainder of the expense.

Under Group 4 purchases were as follows:

Office equipment	$396. 91
Automobiles	20, 896. 79
Trucks	172, 997. 61
Tractors	101, 874. 68
Wagons and harness	$2, 434. 85
Road machines	69, 727. 27
Other machinery and equipment	294, 327. 45
Livestock	6, 261. 00
Land	2, 267. 03
Buildings, structures, and fences	30, 764. 95
Rifles and guns	169. 35
Books and libraries	3, 306. 65
Miscellaneous (not classified)	3, 027. 50

Group 5 includes all items in all counties which were not clearly classified on the records. In a very few counties it is still considered sufficient to merely record the amounts expended and the recipients, ignoring the value for administrative purposes of a statement of the kinds, classes, and amounts of things purchased. Other counties simply listed various purchases for which no subaccounts had been provided under the general heading "Miscellaneous."

Sufficient detailed records were kept in at least ninety-five of the hundred counties to present a reasonably accurate statement of the general road disbursements for the State.

ROAD MILEAGE

No accurate survey of the local road mileage has ever been made. The survey covered by this report was instituted about August 1 and field work was completed during October. Because of the short period of time available, short-cut methods were of necessity adopted. United States Soil and United States Geological Survey maps were available for almost all counties. These maps, of varying dates, showed all roads in existence at the date of survey. With these maps as a base, parties from the district offices of the State highway commission traversed by automobile each public road in each county, making speedometer records of the length of each, its type of construction, and its classification as A, B, or C according to its general traffic importance in the county system.

In order to preserve some measure of regularity of classification, a form of the type indicated below, defining the three classes of road and indicating the information desired, was issued for the guidance of road-survey parties.

North Carolina cooperative county highway survey for the fiscal year ending June 30, 1930

Major classification of county highways according to traffic	Miles	Percentage of labor, materials, and supplies used for annual maintenance
Class A—highways whose major purpose is to serve intercommunity traffic		
Class B—highways serving in approximately equal proportions farm and intercommunity traffic		
Class C—highways whose major purpose is to serve traffic between farms, mills, camps, etc., and roads of classes A and B		

Each map was reviewed at the central office of the commission in Raleigh. Connecting roads were brought into agreement at county lines and new maps, with a scale of 1 inch per mile, were prepared. Of these maps two are reproduced here for purposes of contrast. Figure 1 is a map of Mecklenburg County, a highly developed community with a close network of county roads. Figure 2 is a map of Clay County, which is sparsely populated and has a relatively undeveloped road system.

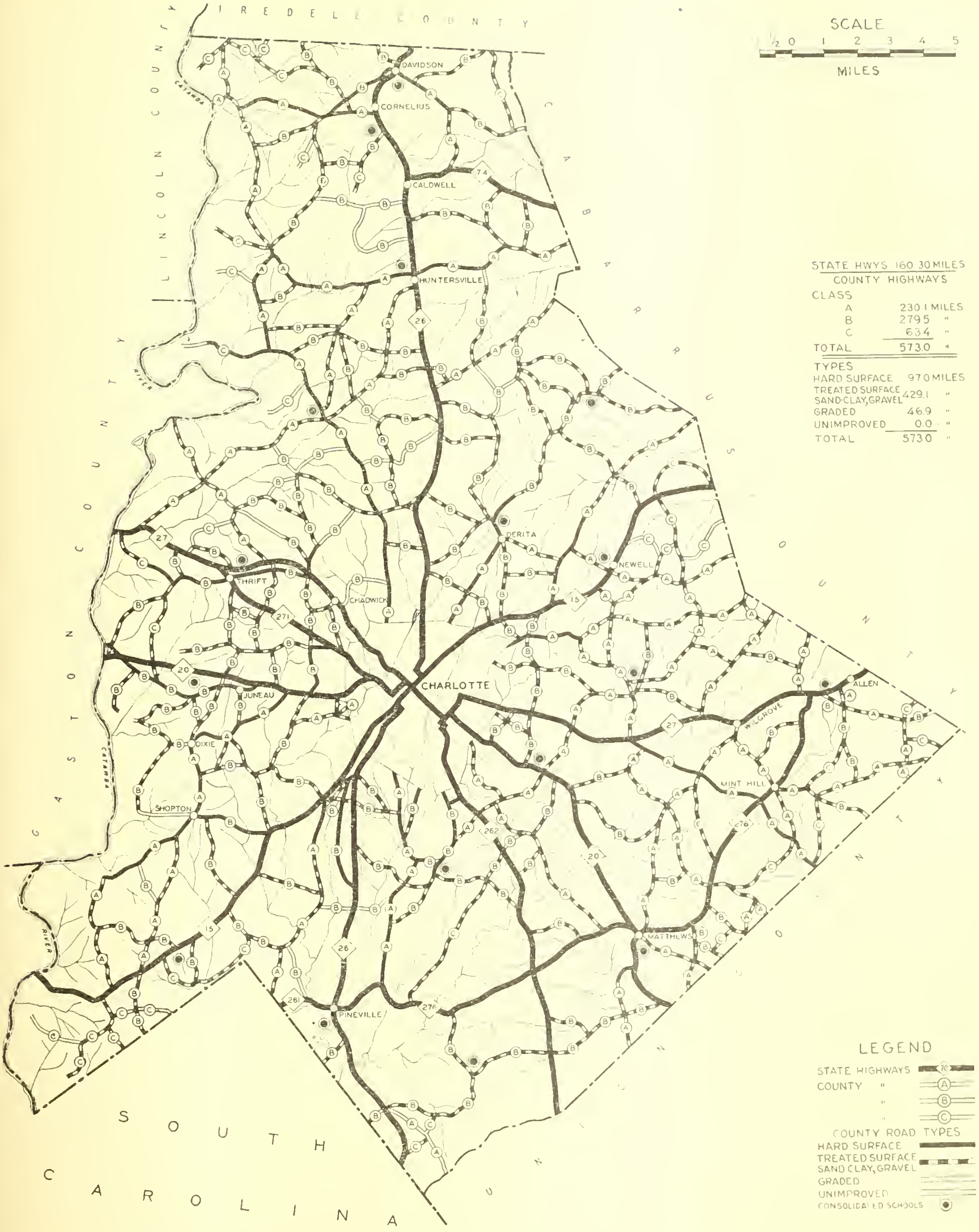

FIGURE 1.—MAP OF MECKLENBURG COUNTY, N. C., SHOWING STATE AND COUNTY ROADS

The survey made in 1926 by the United States Bureau of Public Roads, by visiting each county seat and obtaining estimates from county road officials, revealed a total mileage of 65,311.1. The results of the survey just made show a total mileage of local roads amounting to 45,090.5 Table 4 shows the mileages of each type of road as given by the two surveys.

Table 4.—*Comparative mileage of different types of road as shown by surveys of 1926 and 1930*

Type	Mileage	
	1926 survey	1930 survey
Earth roads, unimproved	20, 359. 0	11, 807. 7
Earth roads, improved	23, 531. 5	19, 056. 8
Sand-clay and top soil	17, 845. 6	12, 260. 2
Sand-clay, etc., treated		335. 1
Gravel, chert, shale, treated and untreated	2, 689. 3	850. 2
Waterbound macadam	237. 2	175. 1
Bituminous macadam	250. 5	137. 4
Sheet asphalt	104. 2	71. 8
Bituminous concrete	24. 5	159. 0
Portland cement concrete	236. 8	203. 7
Brick	32. 5	33. 5
Total surfaced mileage	21, 420. 6	14, 226. 0
Total mileage	65, 311. 1	45, 090. 5

The discrepancy between the two sets of figures may be attributed to two reasons—first, overestimation by local officials in 1926 and, second, transfers of county mileage to the State highway system. These transfers, according to the records of the State highway commission, are as follows:

Type	Miles
Earth roads	995. 05
Topsoil, sand-clay, etc	1, 405. 60
Oil-treated roads (topsoil, etc.)	22. 40
Gravel	64. 95
Macadam	54. 00
Asphalt	65. 25
Concrete	47. 35
Total	2, 654. 60

With a discrepancy of 20,220.6 miles between 1926 figures and the results of the 1930 survey, with 2,654.6 miles accounted for by transfer, it is apparent that the 1926 estimates were made on an optimistic basis. These discrepancies in mileage, varying from a few miles to an extreme case, in one county, of 250 per cent of the actual existing mileage as determined by the present survey, indicated on the part of county officials an astounding ignorance of the extent of the road mileage under their jurisdiction.

During the progress of the survey estimates were made by the field parties of the amount of construction and reconstruction performed by the counties during the year just closed. County mileage constructed and reconstructed during the fiscal year 1930 is shown in Table 5. The mileage improved, as stated above, represented 6¼ per cent of the total county mileage.

The classification of the roads into classes A, B, and C is admittedly an approximate segregation. However, in the absence of a complete traffic survey, it was believed that by this effort a classification which would be of some value in determining traffic flow would be secured. The results of this segregation and the estimated amounts expended for maintenance of three classes of roads are given in Table 6.

The sums expended for maintenance were obtained by deducting from the total, expenditures for construction, financing, etc., which left a sum of $6,567,146.86, which, for the purposes of the survey, was classed as a maintenance cost. It is more than probable that some of the reconstruction or betterment costs, especially for work done on topsoil and sand-clay roads, are included in this sum. However, there were no data available from which to make such corrections; and for the purpose of obtaining the ratio of maintenance expenditures on the three classes of road, the effect of the inclusion of some items of expense for purposes other than maintenance will be small.

Table 5.—*Mileage of roads constructed and reconstructed by the counties of North Carolina during the fiscal year 1930*

Type	Constructed	Reconstructed	Total
Improved earth	699. 5	408. 5	1, 108. 0
Sand-clay and topsoil	619. 3	785. 8	1, 405. 1
Sand-clay and topsoil, oil treated	16. 2	12. 7	28. 9
Gravel, chert, etc	126. 9	84. 6	211. 5
Macadam	7. 9	45. 0	52. 9
Bituminous macadam		3. 6	3. 6
Bituminous concrete	1. 0		1. 0
Portland cement concrete	9. 3		9. 3
Grand total	1, 480. 1	1, 340. 2	2, 820. 3

Table 6.—*Classification of North Carolina county roads according to traffic importance and analysis of maintenance expenditures on each class*

Class	Miles	Per cent	Annual expenditure for maintenance	Per cent	Average expenditure per mile
A	11, 375. 2	25. 2	$2, 343, 133. 28	35. 8	$207
B	17, 477. 0	38. 7	2, 557, 710. 79	38. 8	146
C	16, 238. 3	36. 1	1, 666, 302. 79	25. 4	102
Total	45, 090. 5	100. 0	6, 567, 146. 86	100. 0	146

The computation of average maintenance expenditures per mile assumes that the total expenditure was spread over the entire mileage of each class of roads. As previously stated, observations made during the course of the survey indicated that there were many miles, particularly of the less important roads, that apparently had not received the benefit of any systematic maintenance expenditure. The figures given in the table as average expenditures are therefore theoretical; and, as there was probably some mileage of each class upon which during the year there was actually no expenditure, the true average cost of maintenance for the mileage maintained would be somewhat higher than the theoretical averages.

With this explanation, the table shows that class A mileage, including the important county arteries, comprises about one-fourth of the county mileage. These roads should carry the greatest traffic and do receive a higher expenditure per mile for upkeep. It would follow in a well-coordinated road-improvement plan that the improved mileage would be found in classes A and B, as the total of surfaced mileage (14,226) is some 2,850 miles in excess of the total of the A group. However, as county boards are not permanent bodies, the plans of one board are not binding on their successors, and with each member of the board free to develop policies of the subdivision of the county represented by him, the road maps reveal a considerable mileage of class A road lacking improvement and a corresponding percentage of class C mileage which has been improved. This condition occurs throughout the State and is the inevitable result of the lack of definite program and the expression of the individual judgment and preference of members of the county boards.

No attempt was made to determine the miles of the various types of roads appearing under the A, B, and C classifications, as time and sufficient personnel were not available. County roads, in general, are not built to the standards set by the State Highway Commission either in width, alignment, or grade. Of course, some exceptions may be found, but of the county mileage being taken over by the State, the greater percentage requires widening, easing of curves or actual addition of material.

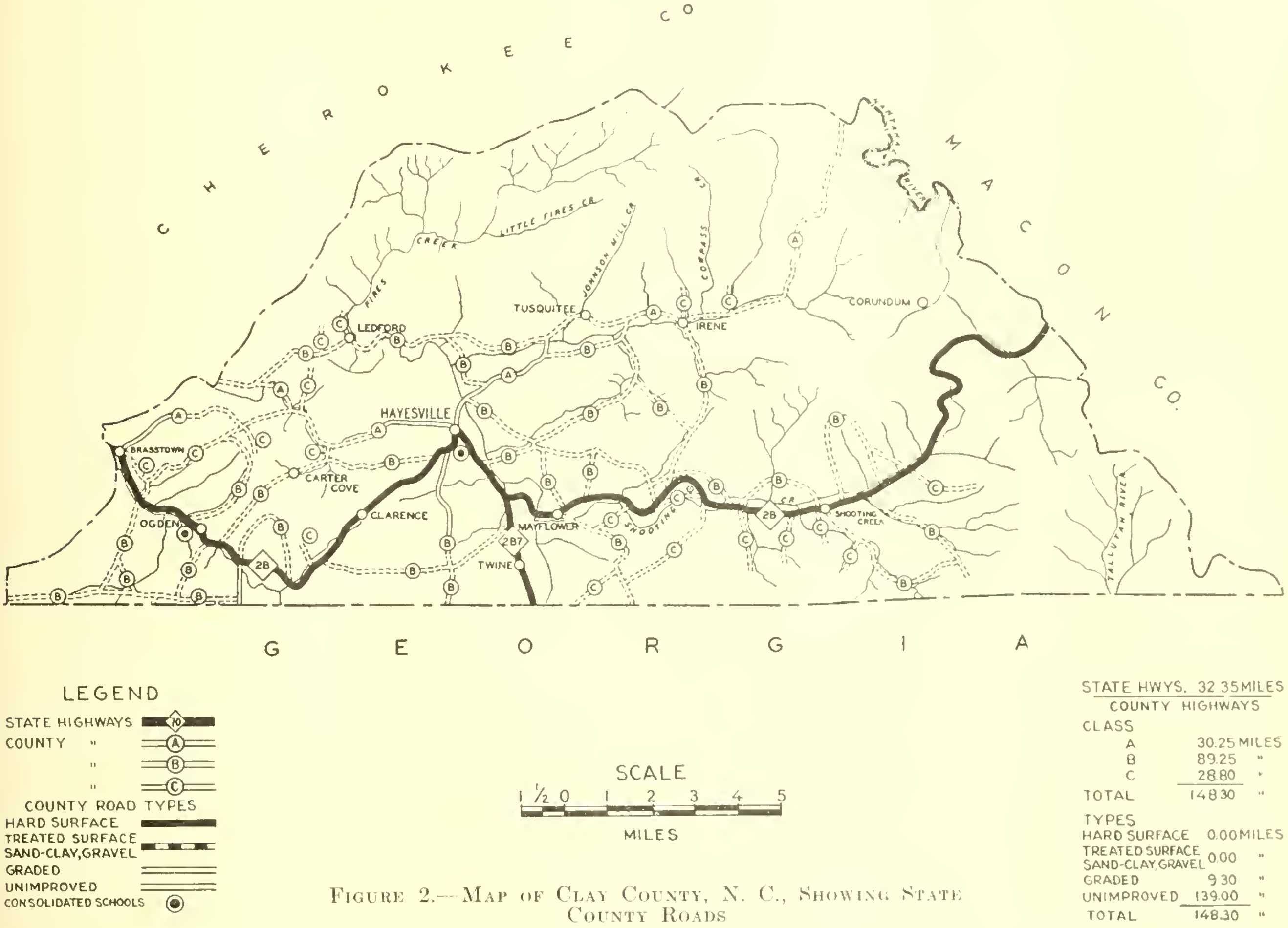

Figure 2.—Map of Clay County, N. C., Showing State County Roads

Future construction by the counties will be of limited mileage annually for a number of years because of shortage of funds. Bond issues are few and small in amount, except for refunding purposes, and public sentiment, especially at this time, is unfavorable to an increase of road taxes.

County maintenance in 47 counties is handled by convict gangs and in 53 counties by hired labor. In general, roads falling in class A receive the first attention from the maintenance forces, with roads in class C receiving whatever funds may be left after the A and B roads are attended to.

With a large mileage of earth roads, improved and unimproved, and with topsoil and sand-clay the predominant types of surfacing, road maintenance becomes largely an operation of cleaning ditches and dragging surfaces. Labor utilized after cleaning and dragging have been performed at necessary intervals becomes labor more or less wasted as far as lasting improvement is concerned, and this aspect of convict utilization offers a fruitful field for future study. Some procedure should be developed to secure greater utilization of convicts on permanent improvements than is now being done in many counties.

RÉSUMÉ OF FACTS DISCLOSED BY SURVEY

County road laws and procedures in North Carolina have not kept abreast of the actual changes occurring in volume and kind of traffic developed since 1868, at which time the present constitution of the State was adopted. For many years subsequent to the adoption of the constitution, horse-drawn traffic, the sole user of the highways, was small in amount and the radius of travel was short. Roads therefore were a local problem, used by the adjacent residents, and their condition was a matter of no concern to the residents of other sections of the township or county. Administrative road units comprising a township were better suited to local road needs in 1868 than they are at present, when automobiles may cross a township in 15 minutes and some counties in about a half hour. Rural highways are no longer a township matter, and, as this survey indicates, at least 25 per cent of the rural roads outside of the present State system are of intercounty importance. Again, township road officials, executing their offices as part-time occupants, with their regular vocations requiring their best efforts and attention, are not qualified to meet modern traffic needs and require-

Table 7.—*Estimated expenditures for public road maintenance by the counties of North Carolina ranked in groups of 10 according to assessed valuation*

Counties	Per $100 valuation	Per square mile	Per capita	Per mile of road
	Cents			
First ten	15.9	$408	$2.40	$328
Second ten	28.0	211	2.40	178
Third ten	27.0	137	1.92	137
Fourth ten	22.8	101	1.86	117
Fith ten	27.8	102	1.94	104
Sixth ten	29.8	93	1.92	104
Seventh ten	25.0	67	1.61	75
Eighth ten	29.6	60	1.90	95
Ninth ten	25.4	62	1.66	75
Tenth ten	32.6	44	1.94	90
Average	22.1	135	2.04	145

Table 8.—*Estimated expenditures for public road maintenance by the counties of North Carolina ranked in groups of 10 according to land area*

Counties	Per $100 valuation	Per square mile	Per capita	Per mile of road
	Cents			
First ten	26.8	$96	$1.97	$111
Second ten	23.6	144	2.22	158
Third ten	21.4	218	2.51	228
Fourth ten	22.8	103	1.77	113
Fifth ten	24.6	140	2, 06	134
Sixth ten	21.5	71	1.41	82
Seventh ten	15.7	155	1.84	172
Eighth ten	22.0	208	2.38	208
Ninth ten	26.4	106	1.96	108
Tenth ten	18.5	108	1.74	125
Average	22.1	135	2.04	145

Table 9.—*Estimated expenditures for public road maintenance by the counties of North Carolina ranked in groups of 10 according to population*

Counties	Per $100 valuation	Per square mile	Per capita	Per mile of road
	Cents			
First ten	16.9	$338	$2.33	$274
Second ten	29.2	199	2.32	175
Third ten	22.2	164	1.95	149
Fourth ten	23.6	93	1.80	99
Fifth ten	27.1	100	1.97	122
Sixth ten	23.4	82	1.70	83
Seventh ten	28.4	98	1.95	117
Eighth ten	27.3	57	1.81	88
Ninth ten	24.5	58	1.71	82
Tenth ten	32.0	47	2.30	93
Average	22.1	135	2.04	145

Table 10.—*Estimated expenditures for public road maintenance by the counties of North Carolina ranked in groups of 10 according to road mileage*

Counties	Per $100 valuation	Per square mile	Per capita	Per mile of road
	Cents			
First ten	21.6	$148	$1.94	$123
Second ten	23.5	209	2.40	172
Third ten	19.3	208	2.36	212
Fourth ten	24.6	134	2.00	142
Fifth ten	20.4	110	1.80	128
Sixth ten	22.8	139	2.13	151
Seventh ten	29.6	89	1.91	102
Eighth ten	24.8	83	1.88	121
Ninth ten	20.5	59	1.60	101
Tenth ten	20.5	67	1.92	167
Average	22.1	135	2.04	145

ments with the full use of modern equipment and procedure found so useful in the construction and maintenance of State highway systems.

COUNTY ROAD EXPENDITURES ANALYZED

It is at least an open question as to whether or not there are too many counties in North Carolina. Ten counties contain 42 per cent of the assessed valuation of the State, 20 contain 56.7 per cent, 30 contain 67.8 per cent, and 40 contain 76.5 per cent. This situation reveals that 60 counties contain only 23.5 per cent of the wealth, with the result that fixed charges necessary to maintain a county seat and the officials required by law imposes a tax upon property far greater in percentage than in the counties of the first groups. The effect of this condition is shown in the analysis of the road problem given in Tables 7, 8, 9, and 10. In these tables the 100 counties are ranked in groups of 10 on the basis of the four major factors, assessed valuation, area, population, and road mileage; and the estimated expenditures for public road maintenance are analyzed in relation to these four factors. The same total of $6,567,146.86 was used in these comparisons as was used elsewhere in this report as being the sum actually expended for upkeep of existing roads.

These analyses show that per capita expenditures are highest in the richest and poorest counties, while the tax on property is at its lowest rate in the richest counties and highest in the poorest counties. It is apparent that a low tax rate in the wealthy counties produces sufficient funds to carry out maintenance with a surplus for construction, while, on the other hand, the poor counties are compelled to raise their rates in order to provide for the absolutely necessary road-maintenance requirements.

CONVICT CAMPS AND ROAD EQUIPMENT

Two of the largest items of cost in counties that employ convicts are the maintenance of convict camps and road equipment. The relation between the cost of maintaining the convict camps and the mileage of roads in a county, and the similar relation between the value of machinery owned and mileage of road, may be considered to represent the relative sufficiency of the convict establishment and machinery owned. A county in which these ratios are high may be considered as relatively fully equipped; one in which they are low must be considered as less fully equipped in respect to both convict forces and road machinery. There is in the various counties a wide variation in this relationship, indicating a very considerable variation in the sufficiency of the labor supply and machinery equipment for the road maintenance requirements of the several counties.

Thus, in New Hanover County the cost of convict camps per mile of road in 1930 was $326.70, while that for Harnette County was $2.44. The average cost of convict camps for all counties using them was $47.27. Among the counties using convict camps, New Hanover County showed the highest value of machinery owned per mile of road, $398; Duplin County, with $14.90 per mile, showed the lowest value. The average value of machinery owned per mile of road by counties using convict camps was $69.30. Among the counties not using convict camps, Onslow County had a value of machinery owned of $181.50 per mile of road; Alleghaney County had only $3.03 per mile. The average value of machinery owned per mile of road by counties not using convict camps was $39.30.

It is not necessary to go into the question of what constitutes adequate equipment. It is sufficient to point out that, as the character of the work done in the various counties in North Carolina does not differ greatly, certain counties are obviously much more fully equipped to do the work than others. Some may be overequipped; some may be underequipped. The investigation that has been made has not been sufficiently complete and searching to determine with exactness the optimum equipment in any case; but the facts gathered do indicate that there is wide variation in relative adequacy of such equipment.

In the relations of convict camp costs and machinery value to assessed valuation of property, a similar wide variation in the ratios for the several counties was observed. Again, it is not necessary to consider at this time what would be a reasonable ratio; it is sufficient to note the variation in these ratios, which may be considered as measuring the relative weight of the burden represented by the cost of convict camp maintenance or machinery ownership, as the case may be.

In Buncombe County the cost of convict camps per $100 assessed valuation was 10.5 cents; in Cleveland County only 1.6 cents. The average cost of convict camps was 5.6 cents per $100 assessed valuation. Among those counties using convict camps, the value of road machinery owned in Scotland County was 24.2 cents per $100 valuation, while that in Gaston County was valued at 2.0 cents per $100 valuation. The average value was 8.2 cents. Among those counties not using convict camps, Onslow County, with 75.0 cents, showed by far the highest value of machinery owned per $100 valuation; while Mitchell County, with 0.1 cents, showed the lowest value. The average value for this group of counties was 10.5 cents per $100 valuation.

It may be further observed that in certain counties the burden of machinery ownership is relatively heavy, although the value of machinery owned per mile of road is relatively low; and that in other counties in which the value of machinery per mile is relatively high, the burden represented by such ownership is relatively low. Similar comparisons may be made in respect to the cost of convict camp maintenance.

REDUCTION IN NUMBER OF COUNTIES DESIRABLE FROM ECONOMIC VIEWPOINT

In the conditions brought to light by these comparisons lies the crux of one of the most serious problems of local highway administration in North Carolina, as in other States. It is clear that the root of the difficulties springs mainly from two facts: (1) That the county unit is not sufficiently large to include areas of rich and poor development within the same administrative borders; and (2) that the area of the average county and the extent of its road mileage are not sufficient to permit of full utilization of the force and equipment required for economic highway operations.

Thus, the small, wealthy county is not much more favored than the large and poor county. Though the latter is likely to suffer economic loss by reason of the poverty of its equipment and resources, the former is almost equally susceptible to loss by reason of the incomplete utilization of a rich endowment.

Considerations of this nature apply not only in the field of highway operations but in all other phases of county administration as well, They suggest that it would be desirable that study be made of the need for continuance of the present number of counties. With the present facilities for travel provided by the State highway system, at least half of the present number of counties could be consolidated and the residents of the larger counties would be in a closer touch with the county seats than the inhabitants of the present counties were ten years ago.

With respect to highway administration, similar advantages would accrue from a pooling of convict forces and machinery for the common use of several counties comprising a group of sufficient size to permit of an equalization of the financial burden and a more complete utilization of resources. As a practical measure it appears that the grouping represented by the 20 judicial districts of North Carolina merits careful consideration.

In the fact that county authorities were so unfamiliar with the highways under their control as to have reported in 1927 a total mileage exceeding by over 17,000 miles the mileage discoverable in 1930; and in the further fact that such improvements as have been made have been distributed apparently with little regard to the relative importance of the highways; in these facts revealed by the survey there is more than in intimation of inefficiency in county administration as at present organized.

In the enormous total expenditure for gasoline, oil, and grease there is strong evidence of waste and maladministration. In the practice of bonding for ordinary annual expenses of maintenance and repairs, there is strong evidence of the need of supervision by competent financial authority.

It is also evident that a part of the cost of State and county law enforcement is being paid under the guise of road expense. There is no reasonable excuse for the payment of criminal court costs from highway funds. Whether or not it is possible, in view of the constitutional limitation upon property taxation, to correct this anomaly is a question that deserves consideration.

Ad valorem taxes collected during 1930, including current and delinquent taxes, imposed a burden for the State as a whole of 19.3 cents for $100 valuation. Ninety counties imposed rates of less than 35 cents, while seven counties levied taxes at rates ranging from 39 cents to 75 cents per $100. These rates, in general, are very moderate and the tendency to the higher rates is observed in those counties whose necessity for existence may be questioned.

Additional funds, if required for local road purposes, should be secured by additional local taxes. The State gas tax and motor vehicle fund has been reduced to the point where further diversions of funds to local purposes would endanger the present State plan of financing, constructing, and maintaining the State highway system. The present gasoline tax rate is now 5 cents per gallon, and this approaches the refinery price of gasoline. Additional gasoline taxes if imposed would tend to become a sales deterrent, probably reducing the income derived from that source.

RECOMMENDATIONS

1. It is recommended that the present law giving counties the option of using their allocation of gasoline tax for general county debt service or for road purposes be amended so as to limit the use of such funds to road purposes. This law now requires that expenditures of the proceeds of the 1-cent gasoline tax for local road purposes be made under the supervision of the State highway commission. The gasoline tax is imposed upon motorists to provide for highway improvement and upkeep, and a diversion of these funds to other purposes should not be made.

2. Recent additions of county roads to the State highway system have created a condition whereby the State highway commission, with its present supply of funds, will not be able for many years to improve in accordance with traffic requirements the mileage now under its control. Further additions to the present State highway system or diversions from the State highway fund to other purposes should not be made.

3. This investigation reveals that under the present county and township organizations road funds are, in many instances, expended without regard to the traffic importance of the particular roads improved. To remedy this condition provision should be made for a classification of local roads on the basis of traffic surveys and other investigations by State and local officials working in cooperation, and for the development of a financial plan to provide for the progressive construction and annual maintenance of the several classes consistent with the resources available and the traffic importance of each class. Pending the development of such a plan, there should be no increase of taxation for local road purposes.

4. Machinery purchases, operation, and upkeep for local roads are the occasion of much uneconomical expenditure. Some adequate State supervision and control should be established so that necessary machinery purchases may receive the benefit of the reduction in price to be obtained by group purchasing, that the selection of machinery may be more in accord with the actual needs, and that large units of equipment needed for only occasional work may be moved from one point to another, to the end that waste by idleness of this class of equipment which now exists can be avoided.

5. It is apparent that efficient utilization of the present county convict forces presents great difficulties, especially in the ordinary maintenance of the roads, and it is likewise apparent that there are few counties that will have sufficient money available for construction purposes in the immediate future to give profitable employment throughout the year for their convict forces. It seems necessary, therefore, that in order to work county convicts economically they must be divided under two general groups, so that those who are classed as "honor prisoners,"—those who can be worked without guards—can be utilized in ordinary maintenance work by the counties; and that those prisoners who have to be worked under guard, and therefore in larger groups, may be administered under some plan of district organization composed of several counties operated under the control of the State prison. Local road organizations could then obtain from this district camp groups of prisoners from time to time for use in the construction of roads, when funds are available for that purpose; and the district prison camp could develop such other lines of work as would supplement the road work that was available.

6. All township and special road district organizations should be abolished at an early date, and all control over the roads now under their supervision should be vested in organizations of wider territorial jurisdiction.

7. Additional powers should be vested in the North Carolina County Government Advisory Commission over budget and accounting affairs in the counties. At present these powers are advisory. The addition of supervisory and regulatory powers to this commission would be of great value in further standardizing financial procedure and safeguarding public expenditures.

8. Plans should be formulated whereby a State purchasing agency may serve the local road organizations in the standardization and purchase of road equipment and supplies.

9. Finally, it is recommended that the State highway commission be charged with general supervision over improvement and maintenance of the roads classed as of major importance under the classification of local roads on the basis of traffic surveys, as proposed in paragraph 3.

TRUCK OPERATION AND PRODUCTION IN CONCRETE PAVING WORK

By Andrew P. Anderson, Highway Engineer, Division of Management, U. S. Bureau of Public Roads

CAREFUL studies on more than one hundred Portland cement concrete paving projects throughout the United States show that an average of 17 per cent of the time during which the construction crew is out on the road ready to work is actually lost because of insufficient supply and faulty operation of the hauling equipment. The function of the hauling equipment on a concrete paving job is simply to transport the batches from the material yard to the mixer skip at the rate at which they may be utilized by the paver. To perform this function effectively should not be very difficult; and yet its inefficient conduct is probably the reason for more losses in profits and more failures of contractors than any other one thing.

In order to obtain a better insight into this rather general difficulty and, if possible, to develop some concrete evidence as to how and to what extent these time losses may be reduced or eliminated, an intensive analysis was made of the accumulated field records of the production studies which have been made on actually going projects by the engineers of the division of management of the Bureau of Public Roads. These records now contain fairly complete data on more than a hundred concrete paving jobs on which trucks formed the hauling equipment and on each of which the studies cover a period of from one to three months. They are of sufficient volume, therefore, to be reasonably representative of present general practice under actual field conditions. The more pertinent facts disclosed by the analysis of these records, and especially by those accumulated during the past three years, form the basis of this article.

CONCLUSIONS OUTLINED

These facts seem to indicate clearly that good management should find it both possible and profitable to reduce materially the interruptions to continuous mixer operation which now arise on the average job, and that it should be both possible and profitable to eliminate by far the greater part of the interruptions arising from faulty hauling operations. While it will no doubt prove profitable to reduce considerably the delays due to occasional shortages in the supply of the hauling equipment, their entire elimination under present methods of operation will probably be impossible without unwarranted expenditures.

There is also ample evidence that the insurance provided against an occurrence of too many of the more expensive mixer delays due to fluctuations in the normal rates of operation justifies a certain apparent oversupply in the hauling equipment. The operation characteristics of the hauling units as well as all auxiliary equipment can readily be determined on any particular job, both as to their individual operation and as to their combined effect on job operation and rate of production; and, when these operation characteristics of the hauling units have been properly ascertained, the number of trucks required to supply the mixer adequately on the varying hauls from day to day can readily be determined.

Other points brought out in this study are that on short hauls the main factor in determining how many batches any given truck can deliver per hour is not primarily the road speed but the length of time the truck must spend in the yard and at the mixer during each trip; and that given equal road speeds and present normal operating conditions, a two, three, or four batch truck does not deliver to the mixer as many batches per hour as two, three, or four 1-batch trucks, respectively. It was found that on 122 concrete paving jobs on which trucks were used for batch hauling, the average length of haul was 2.65 miles. In view of these facts it is evident that the relative production efficiency of single-batch and multiple-batch trucks is one rather important factor which should be given full consideration in the selection of equipment for a given job.

The data indicate further that the present rather common practice of subletting the hauling of the materials on a batch-mile, square-yard, or other similar basis introduces interests which are often antagonistic instead of mutual, in that the largest possible profit from the hauling will be secured when the rate of production is held at a point at which the profits from mixing and placing the batches are actually considerably below their possible maximum.

CAUSES OF LOW PRODUCTION DISCUSSED

In order to obtain a clearer conception of the conditions under which the average concrete paving contractor operates, let us first list the main causes which contribute to keeping down his production. For practical purposes these can be grouped under two main heads: Less-than-capacity load on the controlling or key equipment, and the operation of this equipment at less than its maximum rate. The latter, by far the most common cause of low production, is generally due to various more or less frequent interruptions to the steady, continuous operation of the key equipment. These interruptions or delays are generally termed time losses. In our studies we have for the sake of clarity divided these time losses into two classes: First, those consisting of definite stops each 15 minutes or more in duration, and, second, those less than 15 minutes in duration. The extent of the time losses of the first class as found on the average job is shown in Table 1.

TABLE 1.—*Delays of the first class—Per cent of total available time lost by the mixer in definite stops, each 15 minutes or more in duration*

Cause of delay	Per cent of available time lost
Rain and wet subgrade	17.5
Moving plant set-up	3.5
Lack of materials	3.5
Lack of prepared subgrade	3.0
Inadequate supply and faulty operation of hauling equipment	3.0
Mixer trouble	2.0
Lack of water at mixer	2.0
Loading plant trouble	1.5
Miscellaneous causes	4.0
Total	40.0

After the contractor has obtained his contract, set up his plant, and assembled his crew, we can thus predict that the average job will lose, for various causes, about 40 per cent of the available working hours in definite stops each varying from 15 minutes to days in duration. Nearly one-half of these time losses are each less than one-half day in duration, and thus directly affect both mixer and hauling equipment operation. But this is not the end of the troubles. The records also show conclusively that every job loses a considerable amount of time in minor delays or interruptions, each less than 15 minutes in duration. Furthermore, all these short-time losses occur with the full crew on the job and are thus proportionately much more costly than full-day losses when most of the crew is not paid. The average of these minor time losses based on an analysis of 122 jobs is shown in Table 2.

Table 2.—*Delays of the second class—Average time lost to the mixer for minor delays or interruptions less than 15 minutes in duration, expressed in percentage of the net working time, or total available time minus all time losses 15 minutes or more in duration*

Cause of delay	Per cent of net working time lost	Per cent of total working time lost
Hauling equipment, supply	2.3	1.4
Hauling equipment, operation	7.9	4.7
Hauling equipment, dumping	2.0	1.2
Lack or trouble with water at mixer	3.2	1.9
Subgrade delays	2.7	1.6
Mixer operator	2.1	1.3
Mixer trouble	1.9	1.1
Lack of materials and supplies in place	1.1	0.7
Finishing	0.7	0.4
Miscellaneous	2.1	1.3
Total	26.0	15.6

Many of these second-class delays or minor time losses, and frequently most of them, are of very short duration, but are usually repeated every cycle, so that their total often becomes astonishingly large. Furthermore, all these losses occur with the full crew on the job and are thus proportionately much more costly than full-day losses when the crew is not paid. Table 2 shows that the average job loses 26 per cent of the net working time or 15.6 per cent of the total available time in delays and interruptions none of which are as long as 15 minutes and most of which are each of only a few minutes or seconds in duration but of a constantly recurring nature. In these minor time losses 12.2 per cent of the net working time or 7.3 per cent of the total working time are due to the hauling equipment. These losses are divided on a percentage basis as follows: 19 per cent, or about one-fifth, arise from actual shortage of the hauling equipment; 17 per cent, or about one-sixth, from dumping the batches; and the remaining 64 per cent, or nearly two-thirds, from faulty or inefficient operation of the hauling equipment on hand. It is noted in Table 1 that 3 per cent of the total available time was lost due to the hauling equipment in stops of more than 15 minutes' duration. These naturally all occurred while the full crew was on the job and might better be included under the net working time losses. On this basis the time losses chargeable to the hauling equipment amount to approximately 17 per cent of the net working time, or 10 per cent of the total available time.

CONCRETE PAVING CONSTRUCTION DEMANDS SYNCHRONIZED OPERATION

In order better to understand why it is that the hauling equipment is the cause of such large and persistent delays to the continuous operation of the mixer, it is necessary first to realize how the modern concrete paving job is conducted and what factors or conditions influence or control its rate of production. In a report published in 1925 [1] it was shown that the process of constructing a concrete pavement is essentially a continuing operation in which the maximum rate of production is fixed by the requirements which determine the length of the mixer cycle; and that every interruption to the steady, continuous operation of the paver on this cycle results in a decrease below the otherwise possible rate of production. There is, therefore, practically no opportunity for any appreciable variation in the rate of any of the numerous dependent operations. Nor is there any chance to anticipate the requirements of these different operations, except at the loading plant, where there is usually some opportunity to provide sufficient storage of materials to eliminate the delays which might otherwise occur due to switching or the nonarrival of cars.

From this point on all operations are closely interdependent. The materials must be proportioned, loaded, and hauled to the paver at a rate determined by the duration of the mixer cycle. The operations of finishing and curing can not be allowed to fall behind, nor can those of preparing the subgrade, setting the forms, placing steel, and forming joints be allowed to lag, but all must proceed at least at the general rate set by the mixer.

Losses of time occasioned by lack of coordination between the various operations involved in the paving process are irretrievable. Not even the loss of a single batch from the given maximum rate during one hour can be made up by more rapid operation during the next hour or the next day. If, for example, the mixer cycle is 75 seconds, the maximum possible rate is 48 batches per hour; and a production of, say, 45 batches during one hour can not be made good by a production of 51 batches during the next hour. A batch lost is a batch lost beyond recall.

Such decreases in production would not be a very serious matter were it not for the fact that, exclusive of materials and hauling, the daily or hourly cost of operating a concrete paving plant is practically constant, whether production is at the maximum possible rate or at almost any fraction thereof, so long as it is operating at all. The records which have been examined show rather conclusively that the average hourly cost of operating a modern concrete paving plant, exclusive of materials and hauling, is seldom less than $40 or $45 for every hour the paving crew is out on the road. If, for example, the mixing time and operating requirements permit a mixer cycle of 75 seconds and the hourly operating cost is $40.80, the operating cost per batch will be only 85 cents when the outfit is working at 100 per cent efficiency, and this cost will rise to $1.70 per batch when production falls to 24 batches per hour. This example emphasizes the importance of keeping the mixer in continuous operation on the lowest possible cycle.

Under the usual field conditions neither the mixer nor the hauling equipment can be operated for any extended period of time at 100 per cent efficiency. Theoretically, the mixer can be operated continuously on some definite cycle determined in part by its mechanical condition and the skill of the operator, but mainly by the specifications and working methods required. In practice, neither truck nor mixer operation ever continues for any length of time at this maximum possible rate, and still more seldom do both attain this rate simultaneously. A large number of causes seem to conspire to prevent perfectly coordinated operation. As the rate of maximum production is approached, the difficulties of maintaining this all-essential degree of coordination multiply at an amazing rate. This does not mean that highly efficient operation is impossible, for the records show many cases where for a week at the time operation reached or closely approached 95 per cent of the theoretically possible maximum production.

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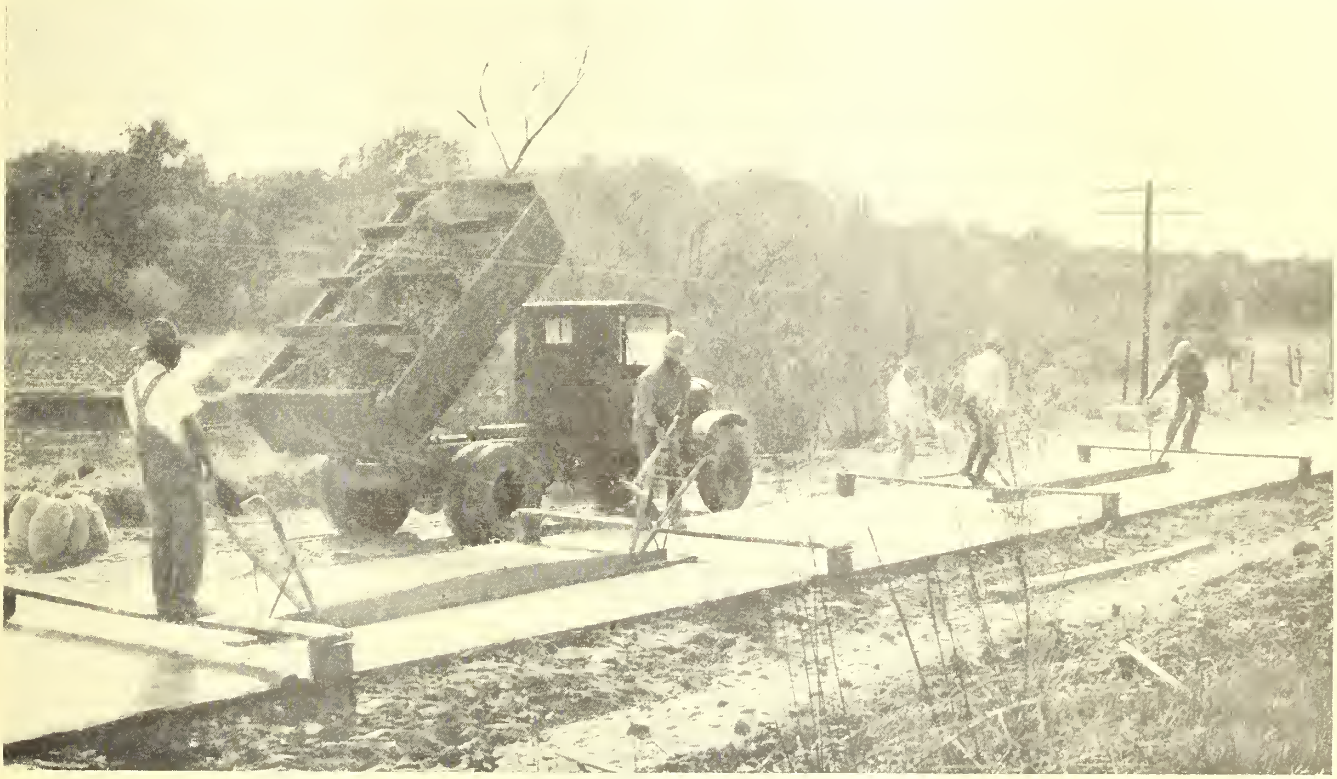
[1] "Efficiency in Road Construction," by J. L. Harrison, Public Roads, Vol. 6, No. 9, November, 1925.



In Order to Maintain a Short Mixing Cycle the Truck Body Must Be Hoisted to Dumping Position Before the Truck Is Backed to the Skip

HAULING OPERATIONS ANALYZED

For the purpose of giving the reader a better understanding of the reasons for those large, persistent, and expensive losses which are due to the hauling equipment, a somewhat detailed analysis will be made of the operating conditions of the hauling equipment and the functions with which it is associated or on which it is dependent. On the average well-managed concrete paving job, the hauling equipment cycle is about as follows: The truck on entering the material yard proceeds to the batcher to receive the sand and coarse aggregate. Usually some maneuvering is necessary in order to get the truck into position under the batcher; especially is this true where more than one size of coarse aggregate is required and two bins are used. If the truck carries more than one batch, it requires some movement before taking on each additional batch. The truck then proceeds to the cement platform where a stop is made to take on the cement bags. If bulk cement is used a canvass cover is usually spread and fastened down after the cement has been dumped on the batches, or a special container is provided. The truck then proceeds to the mixer.

The turntable is usually placed some 200 to 300 feet ahead of the mixer. The truck drives on the turntable, is turned 180°, backs off and proceeds in reverse toward the mixer. If sacked cement is used, it is a common practice for the men who empty the bags ts board the truck at the turntable and empty the bago on top of the batch while the truck is being turned and backed to the mixer. On some jobs a definite stop is made at some other point well ahead of the mixer, where the cement bags are all emptied before the truck proceeds to the turntable. If no turntable is used the turn is usually made somewhat further ahead of the mixer at a place where one or two forms are temporarily omitted so as to provide more room for maneuvering. If the previous truck has been unloaded the oncoming truck proceeds to the mixer, hoists the body to dumping position, and as soon as the skip is down backs into position on the skip. The dump man then drops the first batch; and the truck is driven ahead sufficiently to clear the skip. After the latter has been emptied and returned to the ground, the truck is again backed up and another batch is dropped. When the last batch has been dropped, the truck is immediately driven ahead, the body is lowered, and the truck proceeds to the material yard. On arrival at the yard, the end gates are closed and the truck is ready to begin the cycle of the next load.

TRUCK OPERATING TIME SUBDIVIDED INTO TIME CONSTANT AND HAUL

Truck operation may be subdivided into two parts. The first part includes the time spent while the truck is being loaded at the material yard, plus the time required for turning, backing, and dumping at the mixer, plus such waits or delays as are necessary or incidental to both these operations. The total time required for these operations is designated as the time constant. The second part is the time spent in travel between the materials yard and the mixer, and is designated as the haul. The items going to make up the time constant will be considered first.

An analysis of the various steps comprising the truck-operating cycle shows considerable variations, but an examination of many truck-haul jobs studied during the past three years shows average values as given in Table 3 for the items which go to make up the truck time constant, or the time which the truck spends each trip in other operations than actual driving on the road between the materials yard and the mixer.

Table 3.—*Average truck time constant on all concrete paving jobs*

Operation or item	Time consumed, in seconds			
	1-batch truck	2-batch truck	3-batch truck	4-batch truck
Batcher plant:				
Loading sand and stone or gravel	15	59	114	190
Loading cement	28	80	106	110
Driving and maneuvering within yard	81	99	102	100
Mixer:				
Turning at mixer	68	75	64	93
Backing to mixer	56	77	70	75
Dumping batches	23	128	200	303
Waits or delays	141	178	292	417
Total time constant per round trip	412	696	948	1,288
Total time constant exclusive of waits	271	518	656	871

WELL-MANAGED JOBS SHOW MUCH LOWER TIME-CONSTANTS

While these are the average values for a large number of jobs and thus reflect the operating rates which might reasonably be expected on the ordinary job, they do not show what time is actually necessary to perform these various operations from day to day as indicated under actual field conditions. A smaller number of exceptionally well-managed jobs was therefore selected to afford a nearer approach to the theoretical values of these same items. These are given in Table 4.

Table 4.—*Average truck time constant on well-managed concrete paving jobs*

Operation or item	Time consumed, in seconds			
	1-batch truck	2-batch truck	3-batch truck	4-batch truck
Batcher plant:				
Loading sand and stone or gravel	12	48	90	150
Loading cement	24	47	73	102
Driving and maneuvering in material yard	42	54	60	65
Mixer:				
Turning at mixer	28	39	54	60
Backing to mixer	54	60	75	75
Dumping batches	19	110	157	265
Waits and delays	74	120	161	226
Total time constant per round trip	253	478	670	943
Total time constant exclusive of waits	179	358	509	717

It may be noted first that on the average job, as shown in Table 3, the total time constant is 30 to 50 per cent larger than the net time constant; or, in other words, that on the average concrete paving job about one-third to one-fourth of the actually observed gross time constant is due to waits and delays. Table 2 and the discussion following it show that on these jobs, so far as net working time is concerned, the mixer was operating at an average of about 26 per cent below its full capacity and to this loss of production the hauling equipment contributed about 12 per cent. If we include all hauling delays of more than 15 minutes duration, the figure becomes 17 per cent, or more than half the loss of net working time. This is in sharp contrast to the average time constant on the well-managed jobs. On these the total time constant is on an average nearly one-third lower than on the average job and only about one-fourth of the total time constant is due to waits and delays as compared with about one-third on the average job. Perhaps the most striking difference is the fact that on these well-managed jobs the mixer was operating at an average of only 13 per cent below full production. To this loss the hauling equipment contributed only about 4 per cent or one-fourth as much as on the average job. The records show one job on which during two full months only 1.96 per cent of the mixer time loss was due in any way to the hauling equipment.

DELAYS IN HAULING AND MIXER OPERATION NOT CONCURRENT

Another difficulty is that the operation of the hauling equipment is no more perfect than the operation of the mixer, and sometimes far less. Poor operation of the hauling equipment is quite as likely to occur when the mixer is functioning at its best and therefore requires perfect operation of the hauling equipment in order to avoid delays as it is when the mixer is functioning poorly and does not require so many batches per hour. The net effect of these nonconcurring operating rates results in a bunching of trucks at the mixer when the trucks are operating well and the mixer poorly, and a lack of batches when the conditions are reversed. The extent to which contract hauling tends to aggravate these difficulties will be discussed later.

Since 100 per cent operating efficiency is not long maintained by either mixer or hauling equipment, it becomes necessary to have a certain apparent oversupply of hauling equipment in order to prevent too many mixer delays due to waiting for batches during those periods when the mixer is going well and the trucks poorly. The data seem to indicate that the general practice is to have such a number of trucks that an average of from one to two batches will ordinarily be waiting at the mixer. In practice, this means that at times a dozen, or perhaps even more, batches will be waiting ahead of the mixer, while at other times the trucks will not be quite able to supply its demands. How and to what extent this present practice can probably be improved on will be discussed after we have examined some of the causes which affect truck operation at the batcher plant or material yard.

YARD LAYOUT AND OPERATION ANALYZED

One of the very common causes contributing to a large time constant is a poor material yard layout. Usually there is no choice as to location, but too frequently the best use is not made of the location available. The general practice is to set up the batcher bins so that the trucks must turn and back under in order to receive their load. In fact, many bins are so built that a drive straight through is impossible. This is unfortunate, because turning, backing, and maneuvering a truck requires time which might otherwise be utilized in hauling batches. In fact, the yard layout

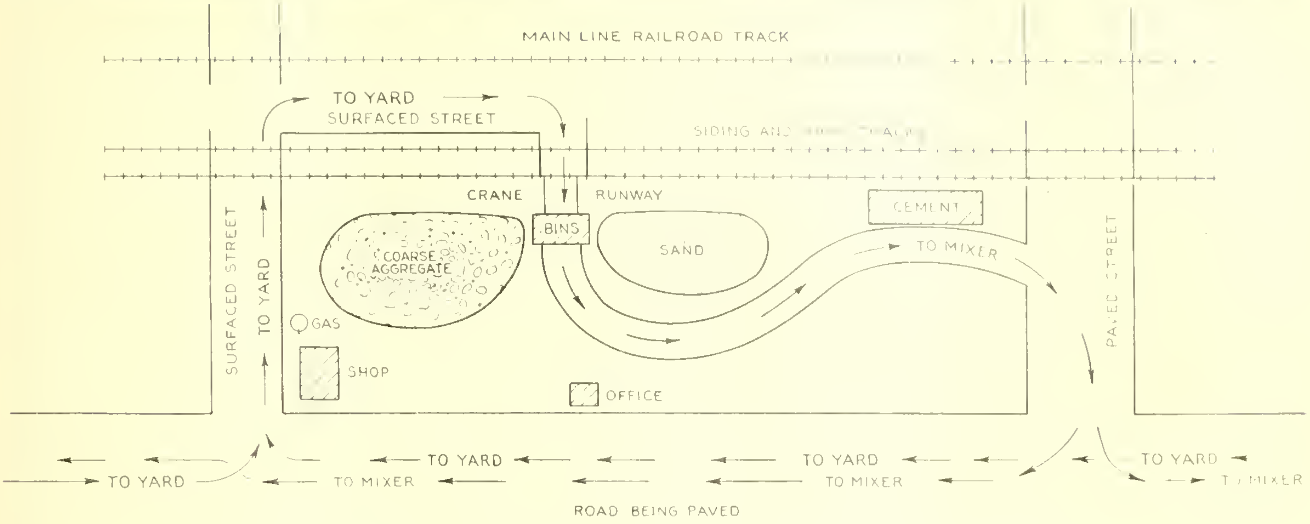


Figure 1. Plan of a Good Yard Layout for Concrete Paving Work

and the routing of the several necessary operations should be planned as carefully as the performance of the several operations at the mixer.

The ideal yard layout would be one in which the trucks could enter at one end on a wide curve, drive straight under a loading bin having compartments for all the aggregates, pass on to a near-by cement-loading platform and storage shed, and then proceed on a tangent or easy curve to the main road. Figure 1 is a sketch of a yard layout which closely approached this ideal. The effect of the yard layout on the time constant is also shown in Tables 3, 4, and 9. In any yard layout the unproductive truck time should be reduced to a minimum, as every deviation from this attainment results in an inescapable penalty in the form of more expensive hauling.

A Sure Way to Get a Large Time Constant: Two Bins, Two Sizes at Each Bin, and Arrangement Such That Trucks Must Be Backed Under Both Bins

The modern bins and batchers can be operated so rapidly and with such regularity that there is usually no valid excuse for requiring the trucks to spend a long time at the loading plant. The modern batchers will ordinarily handle a 30 cubic foot batch of sand and one size gravel in 5 to 35 seconds, depending largely on how dry the sand is. If two sizes of coarse aggregate are used with a 3-way bin, the cycle will ordinarily be increased from 5 to 12 seconds. For 1-batch trucks the actual loading time is practially reduced to the time taken to dump the hoppers, as the filling and weighing can usually be accomplished while the loaded truck is passing out and the empty one is moving into position. Tables 5 and 6 give several values as found from time studies of batcher operation on representative jobs.

Table 5.—*Time of batcher operation as shown by studies of representative jobs*

Type of batching	Operation	Time in seconds
1-compartment batcher	Weighing sand	7.6
	Setting scales	4.1
	Weighing stone	5.7
	Dumping and returning	5.6
	Total cycle	23.0
Sand and 1 size coarse aggregate; 2-compartment batcher.	Weighing sand	7.3
	Weighing stone.	8.9
	Dumping batch.	7.1
	Total cycle	23.3
Sand and 2 sizes coarse aggregate; 2-compartment batcher.	Weighing sand	7.8
	Weighing No. 1 stone	10.0
	Weighing No. 2 stone	7.0
	Dumping batch	5.0
	Total cycle	29.8

Table 6.—*Batcher operation in loading 3-batch trucks by means of a 1-compartment weighing batcher*

Operation	Time in second		
	First batch	Second batch	Third batch
Discharging batch loaded while truck drives in	5.2		
Setting scales		3.0	3.1
Weighing sand		10.9	11.6
Setting scales		2.4	2.9
Weighing gravel		4.2	4.1
Discharging batches		5.6	5.5
Delays and waits, seconds		7.8	
Total time to load 3-batch truck, seconds		66.3	

Unfortunately, the yard layout and operation procedure is seldom such as to permit full advantage to be taken of the rate at which the batchers can be operated. Frequently the yard layout is such that vastly more time is consumed in maneuvering and driving within the yard than in the actual operations of loading. Table 7 is the average record of a large number of time studies on yard operation in one yard. Two-batch trucks were used with a two-bin set-up in a supposed effort to facilitate the use of two sizes of coarse aggregate. A toll of nearly two minutes was exacted from every load which was passed through this yard.

Table 7.—*Effect of poor yard layout on length of time required to load 2-batch trucks*

Operation	Time in seconds
Closing end gates of truck	27.5
Driving to gravel bin and backing under	34.5
Loading gravel, 2 sizes, 2 batches	67.0
Driving to sand bin and backing under	44.0
Loading sand, 2 batches	30.0
Driving to cement car	39.5
Loading cement bags	57.5
Waits and delays	9.5
Total time required for taking on load	309.5

If the Tie Wires Are Cut at the Loading Platform Two Men Can Readily Empty the Cement Bags on the Batches at the Mixer

EFFICIENT HANDLING OF CEMENT ELIMINATES DELAYS

Handling the cement bags sometimes consumes much more time than should be necessary. Twenty-five seconds per batch of seven bags is readily attained on many jobs, though some few of those studied consumed more than twice this much, usually because of improper loading platforms. For speedy loading, the platform should be somewhat higher than the top of the truck body. A little time will also be gained if the bags in the car are stacked to the same height as the number of bags required per batch.

On some jobs, especially in the more arid regions, a practice is made of hauling the sacks to the road on separate trucks and distributing them along the subgrade where they are later picked up by two or three men and emptied in the mixer skip. The records show that when the bags are properly and conveniently placed good laborers can empty them in the skip at the rate of 9 to 10 man-seconds per bag. About the same rate can be attained in dumping the cement bags on the trucks after they arrive at the mixer. Two men are usually employed for this work. The use of bulk cement is rapidly coming into vogue in some sections. Typical time studies of handling bulk cement in 2-wheel buggies show average values as given in Table 8.

The time which the trucks are necessarily delayed at the cement platform need only be a few seconds more than the time required to dump the buggies on the truck, or about 15 seconds per batch. At least one more buggy than the number of batches handled by each truck should be provided.

Table 8.—*Time of handling bulk cement; three men loading buggies*

Operation	Time in seconds
Loading buggy	67.0
Wheeling buggy to scales	13.0
Weighing and adjusting contents	16.4
Wheeling buggy to truck	7.2
Dumping buggy on truck	12.4
Returning buggy for loading	13.0
Total time per buggy	129.0

A few time studies have also been made on the operation of mechanical plants for handling bulk cement. The average values obtained from these studies are given in Table 9.

Table 9.—*Time of operation of bulk cement batcher*

Operation	Time in seconds
Loading and weighing	26.9
Dumping cement on truck	23.8
Total cycle	50.7

Present practice in handling bulk cement usually requires that the cement be dumped on top of the sand and gravel and a canvas cover be spread over the load or else that it be carried in a special container. Spreading and fastening down the canvas covering usually consumes from 20 to 30 seconds additional.

TRUCK OPERATION AT THE MIXER ANALYZED

When the truck reaches the vicinity of the mixer the first operation is usually to turn it either by maneuvering through a space where a couple of forms have been removed, or, more frequently, on a turntable. The value of the turntable lies chiefly in the fact that for some types of truck the time of turning is considerably decreased, and if the ground is soft the subgrade is not cut up so badly. Table 10 shows typical time studies of good operation of a light turntable in turning one-batch trucks.

A 3-Batch Truck Can Be Turned on a Good Turntable in Less Than One Minute

For heavy trucks a larger table is required, and the time elements are somewhat increased, but even a 3-batch truck can readily be turned in 60 seconds.

A rather common custom, where the cement is carried in bags on the batch-trucks, is to have the dumpers climb aboard as the truck reaches the turntable or the turning place. Two men can readily dump the bags

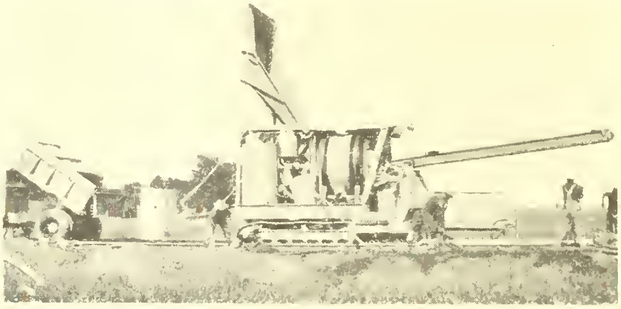
Table 10.—*Time analysis of turntable operation*

Operation	Time in seconds
Running truck on turntable	4.4
Rotating turntable 180°	12.1
Backing truck off turntable	7.7
Total time to turn truck	24.2
Time required to return turntable for use of next truck	10.6
Total operating cycle	34.8

while the truck is turning and backing to the mixer, especially if the tie wires are cut at the loading plant. More frequently a small platform or light truck is kept some distance ahead of the mixer, where each truck stops long enough for the men to step aboard, dump the bags, and step off. The time required is about 5 seconds per bag when two men work on the platform, or 10 man-seconds per bag. While this method makes it easier to take care of the empty bags, as they can all be saved, bundled, and tied up by the men who do the dumping, a definite amount is added to the time constant of the truck operation. For 2-batch trucks carrying 14 bags of cement this is seldom less than 75 seconds. The saving in bags and the added safety of the workers must then be balanced against this extra cost of truck time. Probably there are many jobs on which adoption of this method would prove profitable.

The operation of dumping the batch into the waiting skip usually presents no particularly time-consuming delays except where 1-batch trucks with gravity dump bodies are used. Some types of gravity dump body are very difficult to dump unless the load is placed exactly right, and especially so when the mixer is working down a steep grade. Needless to say, this type of dump body should never be permitted on the job. The actual dumping of properly equipped 1-batch trucks, as well as of each batch of the larger trucks, is often accomplished in 10 seconds, and in no case should the truck detain the skip for more than 30 seconds. An average time of 20 seconds for dropping each individual batch is found on many jobs. The total time which the truck is necessarily detained in unloading, however, varies with the number of batches carried and the operating cycle of the mixer. If the skip is down, the 1-batch truck can drop its batch and immediately proceed on its way for another load. The truck carrying two batches drops the first batch in the skip and immediately, without lowering the body, moves forward sufficiently to clear the skip, waits while the skip is raised and again returned to the ground, and then quickly backs up and drops the second batch. The total dumping time is therefore at least one full skip cycle (usually about 35 seconds) plus the time required to drop the two batches.

Ordinarily the mixer operator will not be ready to raise the skip as soon as the truck clears it after dumping the first batch. For 2-batch trucks, therefore, the total dumping time generally approximates 1½ cycles on a well-managed job on which the practice is to have the truck bodies hoisted into dumping position before backing into the skip and to employ a dump man to drop the batches. The total dumping time of the 3 and 4 batch trucks, however, must include at least one or two full mixing cycles, respectively, plus one skip cycle and the time required to drop the first and the last batch. As in the case of the 2-batch truck, the time actually consumed in dropping the first and last batch will approximate 1½ cycles, so that the time the truck is actually detained even on perfect mixer and truck operation will closely approximate 2½ and 3½ mixer cycles for the 3 and 4 batch trucks, respectively. On the average job the total time the truck is detained at the mixer will be somewhat greater than this because of the delays to continuous mixer operation which occur from time to time, and the more batches carried the more frequently will these delays occur while the truck still has batches to unload rather than when trucks are being exchanged. This fact probably explains why the actual dumping time of the 3 and 4 batch trucks, as shown in Tables 3 and 4, is somewhat longer than would normally be expected when compared with the dumping time of the 2-batch trucks.



The Second Batch Can Be Dropped as Soon as the Skip is Lowered, but a Full Mixer Cycle Must Then Elapse Before the Third Batch Can Be Dropped

HAULING SPEEDS VARY WITH LENGTH OF HAUL AND OTHER FACTORS

Practically all the stop-watch studies on truck operation show the time spent by each truck during each trip in driving from the loading yard to the turning point at the mixer, the time spent in returning from the mixer to the loading plant, and the exact mileage between these points. The condition of the road and the mechanical condition of the trucks is also noted. The road speed of both loaded and empty trucks is therefore readily obtained, as well as the average round-trip speed. The latter quantity should be computed by dividing twice the hauling distance by the total time, loaded and return. The average of the two speeds will not give the correct result, unless they are very nearly equal.

The variations in hauling speeds, both loaded and unloaded are very large. The studies made include jobs on which the average round-trip speed rarely exceeded 10 miles an hour and for individual 1-hour studies was as low as 6 miles an hour, as compared with jobs on which the average round-trip speed exceeded 30 miles an hour and exceeded 40 miles an hour during certain 1-hour studies. The speed was found to vary with many factors, the chief of which were grades, mechanical condition of the trucks, the type of trucks, the amount of traffic interference, whether or not all the trucks were capable of maintaining the same speed, the length of the haul, and particularly the condition of the road surface. The average length of haul from loading yard to mixer of 122 jobs was found to be 2.65 miles. Complete hauling studies are not available for all of these jobs, but the average round-trip speed on all those containing complete data was 18 miles an hour with maximum speeds on individual 1-hour studies running as high as 45 miles per hour to as low as 6.

Table 11, made up from the studies on four separate jobs, illustrates the great variations in average truck speed which are found in the field on different jobs and something of how the average round-trip speed on almost any job varies with the length of haul.

The records bring out three things quite clearly: First, that with good roads, proper equipment in good condition, freedom from traffic interference, and able management, fairly high hauling speeds can be maintained rather consistently; second, that with any one of these requirements lacking the uniform maintenance of high speeds is very difficult and probably impossible; and, third, that a high speed is ordinarily of little productive value unless it can be maintained fairly consistently and includes all the hauling units. Jobs using a variety of trucks of different sizes and speeds show almost without exception a low average speed and a large time constant.

The condition of the road also has a marked effect on the average round-trip speed which the hauling units can and do maintain. Table 12 shows the round-trip speed maintained by 1-batch trucks on an earth road which was generally in good condition, but became spongy and rutted after a rain.

Table 11.—*Data obtained from four fairly typical jobs, showing effect of length of haul on average round-trip speed*

Job No.	Hauling conditions	Length of haul	Average round-trip speed
		Miles	*Miles per hour*
1	Good pavement, 3-batch trucks, excellent condition	1.58	27.1
		1.81	27.5
		2.04	28.3
		4.35	33.7
		4.97	37.2
		6.24	40.2
2	Earth and gravel road, fair to poor; 2-batch trucks, fair condition	.18	6.0
		.46	10.9
		1.17	16.5
		1.73	17.4
3	Earth road, fair condition; 1-batch trucks, fair condition	.52	9.8
		.80	11.4
		1.02	13.1
		1.48	15.8
		1.87	16.5
		2.45	17.1
4	Earth, gravel and pavement, fair to poor; 3-batch trucks, fair condition	.66	7.2
		.95	8.1
		1.20	8.2
		1.78	8.7
		2.08	10.8
		2.30	10.4

Table 12.—*Effect of length of haul and road conditions on average round-trip speed; 1-batch trucks in good condition*

Condition of road	Length of haul	Average round-trip speed	Condition of road	Length of haul	Average round-trip speed
	Miles	*Miles per hour*		*Miles*	*Miles per hour*
Fair	0.35	10	Spongy, rutted	1.80	18
Do	.55	14	Good	2.40	27
Good	.95	21	Fair	2.60	21
Spongy, rutted	1.15	15	Good	2.80	28
Good	1.45	24	Fair	2.90	24
Very good	1.65	31			

HAULING AND RETURNING SPEEDS DIFFERENT

The variation in speed between the loaded and the returning vehicle is sometimes very large. This is especially true on jobs using light 1-batch trucks for hauling a large size batch, thus overloading the trucks sufficiently to reduce materially the speed of the loaded vehicle. Trucks which are not overloaded do not show much variation between loaded and return speeds except when road conditions are poor. Table 13 gives the average hauling speeds on a selection of jobs most of which were operating under better than average conditions. It will be noted that the 1-batch trucks show the widest variations between hauling and returning speeds. This fact is definitely traceable to overloading of these light trucks.

Table 13.—*Effect of loading on speed of trucks in concrete paving work; selected jobs, road conditions good to fair*

Job No.	Number of batches per load	Average speed[1]		Job No.	Number of batches per load	Average speed[1]	
		Loaded	Return			Loaded	Return
		Miles per hour	*Miles per hour*			*Miles per hour*	*Miles per hour*
1	3	17.9	19.5	7	2	17.2	23.8
2	3	18.2	20.0	8	2	17.4	19.8
3	3	23.0	30.8	9	1	11.6	21.9
4	3	23.3	26.1	10	1	16.4	26.9
5	3	24.6	25.7	11	1	17.1	27.5
6	3	25.7	30.7				

[1] Averages of not less than 10 days.

A Few Planks to Bridge a Bad Place in the Road Sometimes Spell the Difference Between Capacity Production and a Complete Shutdown

FORMULA USED FOR ESTIMATING NUMBER OF TRUCKS REQUIRED

A survey of all these data indicates the difficulty of finding and applying any one standard rule or formula as a guide for the proper number of hauling units of any given kind which the contractor should put on his particular job and the manner in which this number should vary from day to day with the length of haul. The present investigation indicates that the only really valuable method is the regular use of the stop watch on the job to evaluate the various factors which make up the time constant and to obtain the actual round-trip speed; and then, by entering these values in a formula or a graph, to obtain the number of trucks which will be needed each following day. This method serves a double purpose: First, the time required to perform each operation under actual operating conditions becomes a matter of definite fact instead of opinion or guesswork; and, second, there is nothing on the job which equals the stop watch for discovering lost motion and wasted effort, and for finding means of improving operating methods. A good stop watch, with a little training in its use and in finding how the various operations should be timed, will undoubtedly prove to be of great value to any road contractor. In no other way can unnecessary time losses and faulty methods of truck operation be so readily detected and the actual effect of new methods determined.

After the various time constants have been evaluated the following formula is suggested for finding the number of trucks needed. The chief merit of this equation lies, perhaps, in its form, which is such that its use should naturally stimulate an interest in each of the various factors which control the number of batches a given vehicle can deliver in a given time. The formula is perfectly general and can be applied to any set of hauling conditions in which the key equipment operates on some definite cycle. It is expressed as follows:

$$N=\frac{120L}{Snt}+\frac{T}{nt}$$

where N is the minimum number of trucks required to keep the mixer in continuous operation.

L is the length of haul in miles from the material yard to the mixer.

S is the average round-trip road speed, in miles per hour, maintained by the trucks between the above points.

T is the total time constant in minutes; i. e., the time which the truck is actually required to spend regularly in loading, turning, backing, dumping, and waiting during each round trip.

t is the actual mixer cycle in minutes; i. e., the time required to pass each batch through the mixer.

n is the number of batches carried by each truck.

The first term of this expression, $\frac{120L}{Snt}$, represents the number of trucks which under these operating conditions should be on the road at any moment, while the second term, $\frac{T}{nt}$, represents the number of trucks which should be either in the material yard or at the mixer.

NOMOGRAM USED TO FACILITATE SOLUTION

The alignment chart or nomogram shown in Figure 2 will frequently prove a timesaver in obtaining the number of trucks for any given set of conditions, and especially in making the numerous solutions required when it is desired to study the interrelated effects of the various factors such as speed, time constant, mixing cycle and the number of batches per load. The scales are logarithmic and equal, except for the middle scale, which is exactly one-half that of the others. The actual scales can therefore be picked off on a piece of drawing paper from an ordinary slide rule. The spacing of the vertical lines is immaterial except that line C should lie equidistant from lines A and E, and B and D, respectively. The vertical position of any two of the scales can be chosen arbitrarily and the position of the others then determined by a single solution of each part of the equation.

The method of solution is shown on the chart. Lines 1, 2, and 3 indicate the manner in which the straightedge is manipulated in the solution of the following problem: Required to find the number of 2-batch trucks capable of maintaining a round-trip speed of 20 miles per hour which are necessary to supply a mixer operating on a 75-second or 1¼-minute cycle when the haul is 5 miles and the total time constant, T, is 7.5 minutes. The number found is 12 plus 3, or a total of 15 two-batch trucks. It will be noted that in the solution of the first term the middle scale is used only to establish a point about which the straightedge is rotated.

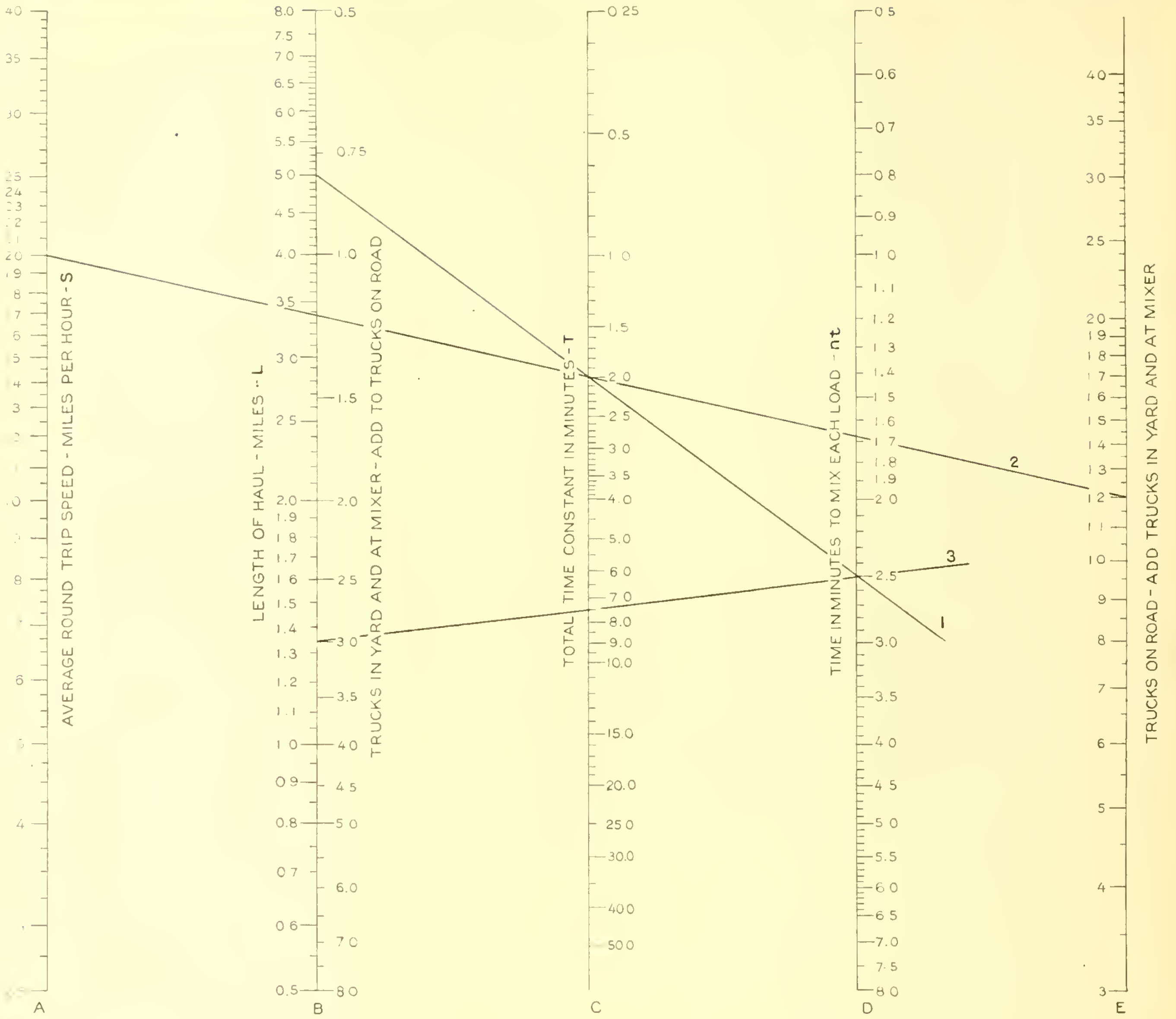
The use of this chart or of the equation with the constants actually observed on various jobs is productive of much valuable information, as is shown by Table 14, which was computed in the following manner. The time constants found as average values on a number of very well managed jobs were used in computing, for various speeds and lengths of haul, the relative delivery ability of multiple-batch trucks in comparison with that of 1-batch trucks. In these computations the mixer was assumed to operate on a 75-second cycle. Computations were made for 1, 2, 3, and 4 batch trucks, for speeds varying from 10 to 40 miles per hour, and for hauls varying from one-fourth mile to 7 miles. For each set of conditions and for the time constants and mixer cycle as stated above, Table 14 gives the number of trucks required to keep the mixer in operation and the relative delivery value of the given type of truck, that of the 1-batch trucks being taken as 1.00 in all cases.

Table 14.—*Number of 1, 2, 3, and 4 batch trucks required to supply mixer and relative delivery value of each in terms of 1-batch trucks when operating under certain similar conditions as to speed and haul distance. Time constants equal 4.2, 8.0, 11.2, and 15.75 minutes for 1, 2, 3, and 4 batch trucks, respectively, mixer operating on a 75-second cycle*

Round-trip hauling speed (miles per hour)	Number of batches hauled per load		Length of haul in miles								
			¼	½	1	2	3	4	5	6	7
10	1	Trucks[1]	5.8	8.2	13.0	22.6	32.2	41.8	51.4	61.0	70.6
		Value[2]	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	2	Trucks	4.4	5.6	8.0	12.8	17.6	22.4	27.2	32.0	36.8
		Values	1.32	1.46	1.63	1.77	1.83	1.87	1.89	1.90	1.92
	3	Trucks	3.8	4.6	6.2	9.4	12.6	15.8	19.0	22.2	25.4
		Value	1.52	1.78	2.10	2.40	2.55	2.65	2.70	2.74	2.77
	4	Trucks	3.7	4.3	5.5	7.9	10.3	12.7	15.1	17.5	19.9
		Value	1.57	1.91	2.37	2.86	3.13	3.29	3.41	3.48	3.54
15	1	Trucks	5.0	6.6	9.8	16.2	22.6	29.0	35.4	41.8	48.2
		Value	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	2	Trucks	4.0	4.8	6.4	9.6	12.8	16.0	19.2	22.4	25.6
		Value	1.25	1.38	1.53	1.69	1.77	1.82	1.84	1.87	1.88
	3	Trucks	3.5	4.1	5.1	7.2	9.3	11.4	13.5	15.6	17.7
		Value	1.43	1.61	1.92	2.25	2.43	2.55	2.62	2.64	2.72
	4	Trucks	3.5	3.9	4.7	6.3	7.9	9.5	11.1	12.7	14.3
		Value	1.43	1.69	2.08	2.57	2.86	3.06	3.19	3.29	3.37
20	1	Trucks	4.6	5.8	8.2	13.0	17.8	22.6	27.4	32.2	37.0
		Value	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	2	Trucks	3.8	4.4	5.6	8.0	10.4	12.8	15.2	17.6	20.0
		Value	1.21	1.32	1.16	1.63	1.71	1.77	1.80	1.83	1.85
	3	Trucks	3.4	3.8	4.6	6.2	7.8	9.4	11.0	12.6	14.2
		Value	1.35	1.52	1.78	2.10	2.28	2.40	2.49	2.55	2.61
	4	Trucks	3.4	3.7	4.3	5.5	6.7	7.9	9.1	10.3	11.5
		Value	1.35	1.57	1.91	2.37	2.66	2.86	3.01	3.13	3.22
25	1	Trucks	4.4	5.3	7.2	11.1	14.9	18.8	22.6	26.4	30.3
		Value	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	2	Trucks	3.7	4.1	5.1	7.0	9.0	10.9	12.8	14.7	16.6
		Value	1.19	1.29	1.41	1.59	1.66	1.73	1.77	1.80	1.82
	3	Trucks	3.3	3.6	4.3	5.6	6.8	8.1	9.4	10.7	12.0
		Value	1.33	1.47	1.68	1.99	2.20	2.32	2.40	2.47	2.53
	1	Trucks	3.3	3.6	4.1	5.0	6.0	6.9	7.9	8.9	9.8
		Value	1.33	1.47	1.76	2.22	2.18	2.73	2.86	2.97	3.09
30	1	Trucks	4.2	5.0	6.6	9.8	13.0	16.2	19.4	22.6	25.8
		Value	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	2	Trucks	3.6	4.0	4.8	6.4	8.0	9.6	11.2	12.8	14.4
		Value	1.17	1.25	1.38	1.53	1.63	1.69	1.73	1.77	1.79
	3	Trucks	3.3	3.5	4.1	5.1	6.2	7.3	8.4	9.4	10.5
		Value	1.27	1.43	1.61	1.92	2.10	2.24	2.31	2.40	2.46
	1	Trucks	3.3	3.5	3.9	1.7	5.5	6.3	7.1	7.9	8.9
		Value	1.27	1.43	1.69	2.08	2.37	2.57	2.74	2.86	2.97
40	1	Trucks	4.0	4.6	5.8	8.2	10.6	13.0	15.4	17.8	20.2
		Value	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00	1.00
	2	Trucks	3.5	3.8	4.4	5.6	6.8	8.0	9.2	10.4	11.6
		Value	1.15	1.21	1.32	1.16	1.56	1.63	1.68	1.71	1.71
	3	Trucks	3.2	3.4	3.8	4.6	5.4	6.2	7.0	7.8	8.6
		Value	1.25	1.35	1.52	1.78	1.97	2.10	2.20	2.28	2.35
	4	Trucks	3.2	3.4	3.7	1.3	4.9	5.5	6.1	6.7	7.3
		Value	1.25	1.35	1.57	1.91	2.16	2.37	2.53	2.66	2.77

[1] Number of trucks required.

[2] Relative delivery value. That of a 1-batch truck is taken in all cases as 1.00



FORMULA FOR COMPUTING NUMBER OF TRUCKS REQUIRED

$$N = \frac{120L}{Snt} + \frac{T}{nt}$$

L = HAUL IN MILES
S = ROUND-TRIP SPEED IN MILES
T = TOTAL TIME CONSTANT IN MINUTES
t = MIXER CYCLE IN MINUTES
n = NUMBER OF BATCHES PER TRUCK
N = NUMBER OF TRUCKS REQUIRED

PROCEDURE

TO OBTAIN FIRST TERM OF FORMULA, PLACE STRAIGHTEDGE TO INTERSECT LINES B AND D AT PROPER SCALE VALUES OF L AND nt, RESPECTIVELY, AND MARK INTERSECTION WITH LINE C; ROTATE STRAIGHTEDGE ABOUT THIS INTERSECTION TO INTERSECT LINE A AT PROPER SCALE VALUE OF S; READ QUOTIENT $\frac{120L}{Snt}$, ON LINE E. TO OBTAIN SECOND TERM OF FORMULA, PLACE STRAIGHTEDGE TO INTERSECT LINES C AND D AT PROPER SCALE VALUES OF T AND nt RESPECTIVELY; READ QUOTIENT $\frac{T}{nt}$, ON RIGHT HAND SCALE OF LINE B ADD THESE TWO QUOTIENTS TO OBTAIN N

Figure 2.—Nomogram for Computing Number of Trucks Required for Given Conditions of Haul and Mixer Operation

LARGE TRUCKS AT DISADVANTAGE ON SHORT HAULS

This table, which is based on the average time constants found on well-managed jobs, seems to indicate that for much of our short-haul paving work the large trucks operate under a rather serious handicap. Thus, on a ¼-mile haul and a round-trip speed of 10 miles an hour, which is quite common on hauls of this length, a 4-batch truck is only worth, in delivering batches, about 57 per cent more per hour than a 1-batch truck, and only about 19 per cent more than a 2-batch truck. Even on a ½-mile haul and at an average round-trip speed of 15 miles per hour, the 4-batch truck is worth in delivering batches only about 69 per cent more than the 1-batch truck and only about 22 per cent more than a 2-batch truck, which of itself is worth only about 38 per cent more than the 1-batch truck. When the haul has reached an average of about 1 mile, the round-trip speed should be about 20 miles per hour. In this case the 2-batch truck becomes nearly equal in delivery value to 1.5 one-batch trucks, and the 4-batch truck nearly equal to 2 one-batch trucks, or to 1.3 two-batch trucks. On long hauls the apparent advantage of the 1-batch truck at even equal speeds practically disappears.

It should not be inferred from these results that one particular size of truck is necessarily more economical or more desirable for concrete paving work than any other on a given job. The relative cost of hiring or operating the various sizes of trucks must also be considered. For example, on a haul of 1 mile, with an average round-trip speed of 20 miles per hour, and time constants the same as those used in computing Table 14, all trucks would be equally desirable from a cost standpoint if they could be either hired or operated at the following total costs per hour; 1-batch at $1, 2-batch at $1.46, 3-batch at $1.78, and 4-batch at $1.91 per hour, or at any other hourly costs which would maintain this ratio. Similar equality would be attained under the same hauling conditions on a 4-mile haul when the rental or total operating costs of the trucks were as follows: 1-batch, $1; 2-batch, $1.77; 3-batch, $2.40; and 4-batch, $2.86 per hour. These comparisons are based on the assumption that all the trucks maintain the same round-trip speed. If the speeds are different the table can still be used for finding the relative delivery value of any two or more trucks. Thus, if the constants are as above except that the 1-batch trucks can only maintain a round-trip speed of 15 miles per hour as compared with 30 miles per hour for a 2-batch truck, then a 1-mile haul would require ten 1-batch trucks or five 2-batch trucks in order to keep the mixer fully supplied. Under these conditions the delivery value of a 2-batch truck would be equal to that of two 1-batch trucks. Consequently, if 1-batch trucks cost $1 per hour, the 2-batch trucks would be worth $2 per hour in hauling batches to the mixer.

Different values of the time constants, *T*, will naturally modify the values given in Table 14, and in every case the contractor should use the stop watch to determine both the time constants and the speeds at which he can operate under his own conditions, and should recheck these values from time to time. He will then have definite facts on which to determine both the number of trucks required from day to day and the size of trucks which will be most advantageous for his particular requirements.

DISADVANTAGES OF CONTRACT HAULING DISCUSSED

That degree of coordination which must be maintained between the mixer and the hauling equipment with respect to both supply and operation, in order to produce square yards of pavement in place at the lowest possible unit operating cost, can only be achieved if all the operations involved are under the full control of one party. The rather general practice of subcontracting the batch hauling not only divides authority but it sets up two parties with essentially opposing interests, one of which must be subordinated if the job as a whole is to be operated on the most economical plan.

It has been noted that the operating time of the mixer is worth about 75 cents a minute. The time of a truck is ordinarily worth from 2 to 5 cents a minute, depending on the size and type of the vehicle. For both the mixer and the trucks the cost is very nearly constant so long as they are at work, regardless of whether production is high or low. Consequently both the paving contractor and the hauling contractor endeavor to keep their own equipment in steady operation. But the mixer can not operate continuously unless there is an equally steady supply of batches at the skip, and it has been shown that 100 per cent mixer operation over long periods is practically impossible. This means that the rate of demand of the mixer fluctuates; in order to provide for these fluctuations there must be a certain oversupply of trucks. If the paving contractor can eliminate two minutes of lost mixer time per hour by hiring another 1-batch truck he will ordinarily be ahead financially. To the hauling contractor, on the other hand, the provision of this extra truck will mean almost certain loss, since the total waiting time of the trucks will be increased. The usual form of batch-hauling contract calls for compensation on the basis of batch-mile or similar unit. For this reason the hauling contract can be completed at the lowest cost only when no more trucks are employed than can always operate freely without having to wait at the mixer. Nor is there usually any increase in the actual cost of the hauling if the number of trucks is decreased far below that actually required to maintain the rate of the mixer. There is, therefore, no economic incentive for the hauling contractor to have even a full supply of trucks.

Nor is there any great inducement for him to eliminate faulty truck operation which gives rise to delays at the mixer. A minute lost in waiting for the truck to be properly squared around to dump its batch means a definite loss of about 75 cents to the main contractor, but to the hauling contractor the loss will probably not be more than 2 cents if the vehicle is a 1-batch truck, or more than 5 cents in any case. Furthermore, the delays which arise from various forms of inefficiency in truck operation are about four times as large as those which arise from an inadequate supply. Contract hauling, therefore, involves financial risks to the main contractor which should not be assumed without very definite and certain compensating advantages.

These facts indicate clearly that full authority over both truck supply and truck operation should be placed in the hands of the main contractor if the lowest possible cost of producing concrete pavement is to be attained. He is the only one who very materially benefits from efficient truck operation, or who can properly gauge the loss due to an oversupply of trucks as compard with the added insurance which they provide against the more expensive mixer delays. The most desirable amount of truck oversupply will, no doubt, vary from job to job, but so long as mixer delays are caused by truck shortages and the ratio of the value of truck time to mixer time may be as great as 1 to 35, a certain amount will be advisable. For the contractor to own sufficient trucks to supply the longest occasional haul would probably be unprofitable, unless other uses should be available for the extra trucks after completion of the long haul.

In view of the fact that the average haul for all jobs studied in this analysis was 2.65 miles, it would not seem advisable for the ordinary contractor to keep on hand hauldling equipment sufficient to supply the full requirements of the mixer on hauls of more than about 3 miles. Under ordinary operating conditions an average round-trip speed of 20 miles per hour, and a 1-minute mixing specification, a 3-mile haul would require about 18 one-batch, 11 two-batch, 8 three-batch, or 7 four-batch brucks. On the first part of a 6-mile haul (assuming no increase in average speed) this plan would involve the hiring of 3 four-batch, 5 three-batch, 7 two-batch, or 14 one-batch trucks, each hired truck to be released when no longer required. In order to insure getting these extra trucks when needed and maintaining full authority over the control of the drivers, it would probably be necessary to pay a little above the prevailing rental, but this item should not equal the carrying

(Continued on page 260)

HIGH-SPEED PRODUCTION ON ASPHALT CONCRETE PAVING WORK

Reported by R. W. Edwards and N. L. James, Junior Highway Engineers, U. S. Bureau of Public Roads

AN UNUSUAL rate of production in asphalt paving work was maintained on California Federal-aid Project No. 194–A, during the construction season of 1930. On this project, which is situated south of Fresno, Calif., between Malaga and Fowler, and south of the latter, 43,556 tons of asphalt concrete were laid over a distance of 7.54 miles, between August 18 and October 8. The average production for each 8-hour operating day was 1,050 tons. The maximum for any one day was 1,204 tons. The company which did this work has carried its production efficiency to increasingly high standards during the last year. On two previous jobs, at Salinas, Calif., and at Tulare, average production rates of 802 tons and 927 tons, respectively, were established.

That this high rate of production was largely due to efficient management is demonstrated by the results of production studies conducted on this project by the United States Bureau of Public Roads. A scientifically designed asphalt plant of large capacity was selected for the work, together with auxiliary equipment capable of keeping pace with it. Labor and directing personnel were efficient and well paid. Studies of time and production losses on previous jobs enabled the contractors to eliminate such losses on this project. The production studies show that the total loss of available operating time was less than 10 per cent, while the time lost with the crew on the job was less than 3 per cent.

ASPHALT CONCRETE SURFACE LAID ON OLD PORTLAND CEMENT CONCRETE PAVEMENT

The entire 7.54 miles of asphalt concrete surface was laid on an old Portland cement pavement, with the exception of a few changes of alignment which required construction over a new subgrade. For a distance of 4.69 miles, between Malaga and Fowler, the asphalt concrete was laid to a width of 30 feet. The remaining 2.85 miles, lying south of Fowler, is 20 feet wide. The new pavement was constructed in three courses, a base or primary leveling course, used to fill in shoulders, etc., a leveling course, and a surface course. The latter is normally 2 inches thick, while the thickness of the other courses varies considerably. The total thickness of the pavement varies from 9 inches at the edges to about 3 inches at the center, depending upon the irregularities of the old pavement.

PAVING COMPLETED IN 41½ WORKING DAYS

The contract was awarded on July 16, 1930, and the work of clearing and grubbing and providing material for grading the roadbed for the new construction was begun on July 17. Work on culverts and small structures was begun on July 21. Erection of the asphalt plant was begun at Malaga on July 16. The plant erection was completed on August 15, and other preliminary work was sufficiently advanced or completed to permit asphalt paving to start on Monday, August 18. The paving was in progress from August 18 to noon of October 8, a total period of 51½ calendar days including 7 Sundays, Labor Day, and one Mexican holiday. This resulted in 42½ available working days, and out of this one day was lost due to rain or wet grade. The daily average was 1,049.5 tons per actual working day and 1,023.4 tons per available day.

The price received for the asphalt concrete was $4.01 per ton. This price covered all items from furnishing and setting permanent wooden forms to finishing and rolling the pavement. The total for 43,556 tons was approximately $174,660, or $23,164 per mile. The cost per square yard was approximately $1.50.

The general superintendent for the paving company had complete charge of the entire contract and the superintendent of plants was in charge of plant operations. There was a capable foreman for each division of the work, such as hauling, spreading and finishing, setting forms, etc. Two experienced timekeepers were employed to take care of the clerical work and to attend to the ordering of all materials and supplies. These timekeepers enabled the superintendent to devote his entire time to planning and supervision. A carefully planned and detailed schedule of operations was prepared at the start of the job and as a result no delays whatever occurred due to lack of planning or to inefficient management. It was the duty of each foreman to see that his equipment and personnel functioned efficiently and to correct delays immediately. It was through this specialization on the part of the men and coordination on the part of the general superintendent that this job proceeded so smoothly and at such a remarkable rate.

The normal crew per working shift averaged 62 men, which included 16 at the plant, 8 hauling materials to the job, 17 handling material on the job, including rolling, etc., 12 form setters, and 9 on supervision and miscellaneous work.

ADEQUATE MODERN EQUIPMENT USED

The major items of equipment were:

1 new, all-steel, portable asphalt plant, consisting of a 4,000-pound, 80 r. p. m. pug-mill and a 6-foot by 22-foot 8 r. p. m. dryer.
1 1¼ cubic yard clamshell crane (feeding plant).
10 trucks hauling 3 batches or 6.3 tons.
3 9-foot spreader boxes.
1 30-foot finishing machine.
1 20-foot finishing machine.
1 10-ton, 3 wheel gas roller.
3 8-ton tandem rollers.
1 water-tank truck.
4 runabout passenger cars.
1 blacksmith shop.
1 portable field office.

All of the above equipment was of the latest design and in first-class condition.

HANDLING OF MATERIALS DISCUSSED

Materials other than sand were delivered by rail to the plant, located on a siding at Malaga, near the north end of the project. Sand was produced locally and hauled to the plant by subcontract from pits about 8 miles distant. One pit supplied course sand and the other fine sand. It was necessary to premix or blend these sands at the plant to obtain the grading required in the specifications.

The aggregates were unloaded from the cars by the 1¼-yard clamshell crane and either stock-piled or fed into the hoppers at the cold elevator. There were three of these hoppers; one was used for the sand and No. 4 rock, the latter varying from 1¼ to ¾ inch in size. A second hopper was used for No. 3 rock, varying from ¾-inch to rock retained on a No. 3 screen, and a third for No. 2 rock, which was such as to pass a No. 3 screen and be retained on a No. 10 screen. The sand used was such as to pass a No. 10 screen, and included some finer than that retained on a No. 200 screen. Three adjustable gates, regulated by a feeder operator, admitted each size of material to the boot or hopper of the cold elevator. It is interesting to note that each of these materials arrived at the plant in precisely the grading desired in the four hot-storage bins. This was the chief reason for the exceptionally small percentage of bin delays or delays due to lack of proper size of hot aggregate.

Materials after leaving the stock piles passed through the dryer where they were heated to 375° or 400°. They were then conveyed by a bucket elevator to the screens where they were screened and chuted into four hot-storage bins, each bin containing a certain size. All aggregates passed the 1¼-inch screen except part of that for the base course, where a maximum size of 2 inches was used.

Passing from the hot-storage bins, materials were batched by weight in the weigh box, which was suspended on multiple-beam scales, so that each size could be weighed rapidly and accurately. Dust was conveyed from a dust hopper to the weigh box by means of a mechanically driven screw operated by the batcher operator, and weighed on the multiple-beam scales. The dust was not heated. After the materials were weighed, they were dumped into the mixer box by the mixer operator. The gate of the weigh box opened at right angles to the mixer shafts; this arrangement facilitated even distribution of the materials in the mixer. Baffle plates were installed in the weigh box to distribute the materials issuing from the four hot-storage boxes. In order still further to assure even distribution, a definite sequence was adopted for weighing each material.

Asphalt cement, heated to about 280° was weighed and admitted to the mixer, parallel to the mixer shafts, by another operator, who also discharged the mixer and dumped the truck hopper or "gob" box. The mixer gate and truck hopper were operated by steam pistons, controlled by steam valves.

HAULING OPERATION CAREFULLY TIMED

The asphalt concrete, after being weighed by the State highway authorities, was hauled to the job in trucks equipped with tarpaulins. All trucks hauled three batches or 6.3 tons, and all trucks were pneumatic tired. Truck speeds averaged 22 miles per hour. The trucks were turned about 200 feet ahead of the finishing machine and were backed and hooked to the spreader boxes. The contents of each were then dumped on the street ahead of the finishing machine to approximately the desired depth. The terminal time constants of operation for these trucks averaged 6.5 minutes per trip. A careful scrutiny of the time of arrival and departure of trucks from the plant was made by the company to check upon the efficiency of truck operation.

FINISHING MACHINES USED FOR PAVING

A finishing machine was employed to spread the material and strike it off to the desired section. For the 30-foot section a rebuilt machine was used. For the 20-foot section a regular finishing machine, partly rebuilt and speeded up, was used. Studies disclosed that the 30-foot machine would travel at the rate of 7.8 feet per minute and would handle approximately 194 tons per hour, while the 20-foot machine would travel 12.6 feet per minute and handle 205 tons per hour.

Compaction was obtained by the use of four rollers, one 10-ton, 3-wheel roller and three 8-ton tandems. The 10-ton and one 8-ton roller were used for longitudinal rolling and the other two 8-ton rollers were used for transverse or crescent rolling. Late in the afternoon two of the 8-ton tandem rollers were used to roll out bumps.

PRODUCTION STUDIES COVERED BOTH PLANT AND AUXILIARY EQUIPMENT

The studies conducted on this project by the engineers of the Bureau of Public Roads covered the period from September 15 to October 8, and included the following major items:

1. Making several stop-watch studies every day of the operation of the asphalt plant. The purpose of these studies was to determine the amounts and sources of delays at the mixer.
2. Making daily stop-watch studies of auxiliary equipment to determine their rates of production, and to ascertain whether or not they favored or limited the attainment of maximum production by the asphalt plant.

A number of other features of the construction were also studied, including mixing time tests to determine the effect of the length of mixing time on the distribution of aggregates and asphalt in the mixer.

In order that the mixing operation might be carefully controlled during the period when time studies were being made, a timing device was installed on September 3. Although it was an innovation in asphalt work, the use of this device resulted in an almost clocklike regularity of plant operation, a nearly constant mixing time, and approximately a 6.5 per cent increase in rate of production.

For the purpose of the production study, the charging time was taken as the total time consumed in emptying all ingredients into the pugmill. The mixing time was taken as the interval between the completion of the charging and the opening of the discharge gate. The discharge time was taken as the time from the opening of the gate to the completion of discharge. A summary of the results of the plant production study is given in Table 1. It will be observed that for the base and leveling courses the possible number of batches per hour proved to be 73, while for the surface course 61.2 batches were found to be possible. Minor delays brought these rates down to 69.8 and 60.6 actual batches per hour, respectively. These figures are practically the same as the maximum production rates allowed by the State authorities, who would suspend plant operation if production exceeded 70 batches per hour for the base and leveling courses and 60 batches per hour for the surface course. The reason for the difference between the number of batches allowed for base or leveling course and surface course was the fact that the State required a "double charge" on the surface materials, i. e., No. 3 stone was weighed separately and after the weighing and dumping of the other aggregates.

TABLE 1.—*Production data for period of study, September 15 to October 8, 1930*

Item	Base and leveling courses	Surface course and screenings
Charging time, seconds	6. 8	15. 7
Mixing time, seconds	28. 3	27. 9
Discharge time, seconds	14. 2	15. 2
Total cycle, seconds	49. 3	58. 8
Net batches possible per hour	73. 0	61. 2
Per cent of hour lost in minor delays	4. 45	0. 78
Per cent of hour utilized	95. 55	99. 22
Actual batches per hour	69. 8	60. 6
Actual tons per hour of operation	152. 0	131. 0
Average tons per day on days operated	[1] 1, 066. 9	
Average haul to job, miles	5. 57	
Average number trucks for both courses	15. 2	
Actual job efficiency including unavoidable delays	90. 41	
Actual job efficiency excluding unavoidable delays	97. 14	

[1] All courses.

TABLE 2.—*Production data for entire paving period, August 18 to October 8, 1930*

Item	Base and leveling courses	Surface course and screenings	Entire job
Total hours mixing time	208. 80	108. 25	317. 05
Total tons mixed	29, 823. 05	13, 732. 91	43, 555. 96
Total batches mixed	13, 548	6, 373	19, 921
Average tons per batch	2. 20	2. 15	2. 19
Actual batches per hour	64. 89	58. 87	62. 86
Actual tons per hour	142. 83	126. 85	137. 38
Average tons per day on days available			1, 024. 85
Average tons per day on actual working days			1, 049. 50

The production data for the entire paving period are given in Table 2. It will be noted that the actual number of batches per hour was considerably less for the entire job than for the period from September 15 to October 8, when the timing device was in operation.

DELAYS IN PRODUCTION ANALYZED

In Table 3 is given an analysis of the delays occurring during the entire paving period. Time losses are grouped into two divisions, major and minor. Major delays are those 15 minutes or more in length and minor delays are those less than 15 minutes in length. Minor delays usually occur much more frequently than major delays and generally far exceed the major delays in total time even though each may be less than a minute in length. These minor delays are usually the most serious consumers of profits. Major delays fall into two classes: Those occurring with the full crew on the job and those occurring with the major portion of the crew laid off. Delays are classified as avoidable or unavoidable, and the efficiency of the organization is computed exclusive of the unavoidable delays. It is particularly interesting to note that on this project the major delays were all unavoidable and exceeded the minor delays which are believed to be mostly avoidable. The cost and difficulty of further reducing these minor delays would have been great and it is questionable if it could have been done. To illustrate clearly the cost of these time losses, they are evaluated at the rate of $119.18 per hour, which is a conservative value for the normal personnel and organization employed, and the estimated cost is given in Table 3. It should be noted that for the periods when the crew was not on the job the estimated cost of delay was not computed on this basis.

TABLE 3.—*Analysis of delays during entire paving period, August 18 to October 8, 1930. Total available working days during period, 42.50; total available hours, 340.50*

Item	Delay in hours	Per cent of total available time	Estimated cost
Major delays—unavoidable:			
Handling on grade	1. 66	0. 49	$197. 81
Crane repairs	. 50	. 15	59. 59
Plant equipment repairs	4. 62	1. 36	550. 61
Lack of asphalt	. 75	. 22	89. 39
Power off	. 25	. 07	29. 80
Electrical repairs	1. 17	. 34	139. 44
Rain or wet grade (crew off job)	14. 50	4. 25	302. 04
Total	23. 45	6. 88	1, 368. 71
Minor delays—probably avoidable:			
Truck operation	. 22	. 06	26. 22
Lack of hot materials	3. 14	. 92	374. 23
Operative delays	2. 31	. 68	275. 31
Handling on grade	4. 05	1. 19	482. 68
Total	9. 72	2. 85	1, 158. 44
Grand total, all delays	33. 17	9. 73	2, 527. 15

(Continued from page 257)

cost of owning these vehicles during the long periods when they would not be needed on the job.

That both owned and hired hauling units can be combined and operated at a high degree of efficiency when full control is maintained over the hired units is amply demonstrated on several jobs included in this study. On one of these jobs, using partly owned and partly hired hauling units, a rate of over 50 batches an hour was attained during several weeks with a time loss of less than one-half of 1 per cent from faulty truck operation on a haul which at one time exceeded 8 miles. The mixing time was 50 seconds.

CONCLUSIONS OF INVESTIGATION SUMMARIZED

The major point brought out in this investigation is the fact that under present methods and practices of providing, directing and operating the hauling equipment, the paving contractor loses about 17 per cent of his time, which might otherwise be utilized in further production. A part of this loss is due to the rather prevalent use of a method of subcontracting in which the economic interests of the two parties are antagonistic. The use of this system, at least in its present form, is unsatisfactory and should be discontinued. A larger portion of these time losess, however, are due to faulty or inefficient operation. Numerous instances are found where under able management and proper supervision these time losses have been greatly reduced and a corresponding financial reward obtained. Too frequently much of the seeming inefficiency of the truck operation is really due to a poor yard layout or to faulty operation of the loading equipment. Careful planning of the yard layout before the loading plant is set up should obviate many of these difficulties. Once the work of pouring concrete has actually begun, nothing will probably prove so effective in reducing time losses and correcting faulty methods of operation as able supervision abundantly supplied with actual facts obtained through frequent use of the stop watch.

○

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ANNUAL REPORTS

Report of the Chief of the Bureau of Public Roads, 1924.
Report of the Chief of the Bureau of Public Roads, 1925.
Report of the Chief of the Bureau of Public Roads, 1927.
Report of the Chief of the Bureau of Public Roads, 1928.
Report of the Chief of the Bureau of Public Roads, 1929.
Report of the Chief of the Bureau of Public Roads, 1930.

DEPARTMENT BULLETINS

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*691D. Typical Specifications for Bituminous Road Materials. 10c.
1216D. Tentative Standard Methods of Sampling and Testing Highway Materials, adopted by the American Association of State Highway Officials and approved by the Secretary of Agriculture for use in connection with Federal-aid road construction.
1279D. Rural Highway Mileage, Income, and Expenditures 1921 and 1922.
1486D. Highway Bridge Location.

DEPARTMENT CIRCULARS

No. 331C. Standard Specifications for Corrugated Metal Pipe Culverts.

TECHNICAL BULLETIN

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MISCELLANEOUS CIRCULARS

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No. 76MP. The Results of Physical Tests of Road-Building Rock.

SEPARATE REPRINTS FROM THE YEARBOOK

No. *914Y. Highways and Highway Transportation. 25c.
937Y. Miscellaneous Agricultural Statistics.
1036Y. Road Work on Farm Outlets Needs Skill and Right Equipment.

TRANSPORTATION SURVEY REPORTS

Report of a Survey of Transportation on the State Highway System of Ohio. (1927)

Report of a Survey of Transportation on the State Highways of Vermont. (1927)

Report of a Survey of Transportation on the State Highways of New Hampshire. (1927)

Report of a Plan of Highway Improvement in the Regional Area of Cleveland, Ohio. (1928)

Report of a Survey of Transportation on the State Highways of Pennsylvania. (1928)

REPRINTS FROM THE JOURNAL OF AGRICULTURAL RESEARCH

Vol. 5, No. 17, D- 2. Effect of Controllable Variables upon the Penetration Test for Asphalts and Asphalt Cements.
Vol. 5, No. 19, D- 3. Relation Between Properties of Hardness and Toughness of Road-Building Rock.
Vol. 5, No. 24, D- 6. A New Penetration Needle for Use in Testing Bituminous Materials.
Vol. 6, No. 6, D- 8. Tests of Three Large-Sized Reinforced-Concrete Slabs Under Concentrated Loading.
Vol. 11, No. 10, D-15. Tests of a Large-Sized Reinforced-Concrete Slab Subjected to Eccentric Concentrated Loads.

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*Department supply exhausted.

UNITED STATES DEPARTMENT OF AGRICULTURE

BUREAU OF PUBLIC ROADS

CURRENT STATUS OF FEDERAL AID ROAD CONSTRUCTION

AS OF

JANUARY 31, 1931

STATE	COMPLETED MILEAGE	UNDER CONSTRUCTION					APPROVED FOR CONSTRUCTION					BALANCE OF FEDERAL-AID FUNDS AVAILABLE FOR NEW PROJECTS	STATE
		Estimated total cost	Federal aid allotted	MILEAGE			Estimated total cost	Federal aid allotted	MILEAGE				
				Initial	Stage[1]	Total			Initial	Stage[1]	Total		
Alabama	2,177.5	$ 5,219,430.13	$ 2,492,548.80	192.4	27.3	219.7	$ 702,983.97	$ 350,664.72	25.5	3.5	29.1	$ 4,724,072.45	Alabama
Arizona	897.1	3,590,078.21	2,793,599.38	122.0	152.4	274.4	705,392.22	527,811.20	30.9	10.9	41.8	1,852,066.18	Arizona
Arkansas	1,841.1	4,964,099.03	2,304,510.97	161.0	34.7	195.7	1,011,923.15	500,133.65	15.8	13.5	29.3	2,809,350.25	Arkansas
California	1,958.8	7,275,115.23	3,216,358.74	166.2	25.0	191.2	1,695,520.59	791,289.47	39.9	5.9	45.8	3,753,499.51	California
Colorado	1,297.0	3,499,857.27	1,861,935.92	126.0	57.9	183.8	1,345,024.23	755,307.11	64.8	32.5	97.3	3,443,256.71	Colorado
Connecticut	258.2	1,524,709.41	561,789.48	7.5		7.6	1,901,918.93	559,661.99	24.2		24.2	651,955.09	Connecticut
Delaware	318.9	106,632.85	53,316.42	4.4		4.4	434,115.60	216,303.30	12.9		12.9	434,889.13	Delaware
Florida	521.2	5,052,660.24	2,404,264.34	98.8	5.5	104.1	534,834.24	253,166.16	17.6		17.6	2,530,423.87	Florida
Georgia	2,679.7	7,491,514.03	3,576,755.42	199.1	125.1	325.2	2,515,247.57	1,138,080.05	40.9	51.4	92.3	3,505,442.97	Georgia
Idaho	1,295.1	2,124,358.93	1,292,251.73	151.3	12.5	163.8	765,555.18	452,016.46	55.8	7.2	53.0	1,515,888.19	Idaho
Illinois	2,183.4	19,997,652.33	9,092,122.16	571.4	57.3	528.7	1,357,755.66	645,588.15	47.5		47.6	7,133,258.00	Illinois
Indiana	1,579.0	2,090,715.05	1,012,682.46	65.4		55.4	4,835,824.97	1,799,739.32	103.7		103.7	3,897,752.34	Indiana
Iowa	3,229.0	105,290.11	1,892.77		4.1	4.1	7,433,114.40	3,113,533.23	157.9	77.6	235.5	237,794.44	Iowa
Kansas	2,984.9	5,154,094.19	2,434,385.56	276.4	29.6	306.0	2,605,058.09	1,259,270.48	221.8	40.7	262.5	2,840,916.46	Kansas
Kentucky	1,538.5	5,157,002.60	2,317,944.43	176.8	74.0	250.8	2,145,559.83	393,870.13	28.1	22.0	50.1	1,531,097.32	Kentucky
Louisiana	1,427.4	4,312,239.69	2,128,155.36	176.2	8.3	184.5	4,037,806.36	1,785,228.72	86.5	12.7	99.2	812,282.55	Louisiana
Maine	589.1	2,036,579.09	573,182.41	45.8		45.8	972,605.32	375,524.35	7.4		7.4	1,813,909.20	Maine
Maryland	693.1	645,177.07	297,189.33	5.4	7.3	13.7	291,945.10	106,661.80	4.9	2.4	7.3	932,519.59	Maryland
Massachusetts	711.8	8,271,609.92	2,125,108.27	73.3		73.3	381,271.27	55,400.00	3.8		3.8	2,469,043.29	Massachusetts
Michigan	1,788.3	7,857,015.97	3,297,669.03	205.4	22.9	228.3	1,578,095.94	627,937.17	37.5	12.8	50.3	4,018,208.09	Michigan
Minnesota	4,317.4	2,041,829.15	768,564.13	22.3	92.9	115.2	5,151,384.55	1,987,745.64	54.5	159.3	213.9	980,922.58	Minnesota
Mississippi	1,816.5	1,034,435.33	513,527.80	35.6	31.4	67.0	459,787.66	229,893.82	11.1	21.1	32.2	5,273,159.91	Mississippi
Missouri	2,598.2	8,426,700.95	3,019,508.08	145.2	49.8	195.0	2,948,053.18	1,332,209.13	103.1	28.0	129.1	2,214,961.53	Missouri
Montana	1,892.0	7,343,731.48	4,144,714.08	573.6	40.9	514.5	2,166,732.30	1,211,963.59	158.8	26.4	183.2	4,098,860.13	Montana
Nebraska	3,852.9	4,754,108.66	2,290,577.28	268.3	69.8	338.1	1,572,253.53	669,379.65	45.1	22.8	57.9	3,649,467.21	Nebraska
Nevada	1,278.0	1,005,734.41	871,720.23	52.4	63.4	115.8	10,142.81	9,011.88		.8	.8	1,881,583.08	Nevada
New Hampshire	382.1	595,295.53	222,027.15	9.8		9.6						572,295.18	New Hampshire
New Jersey	554.8	2,855,418.71	758,991.32	19.5		19.5	822,104.55	350,793.09	12.8		12.8	2,197,518.57	New Jersey
New Mexico	2,008.8	3,470,620.18	2,298,222.40	138.7	40.5	179.2	965,711.60	572,588.97	30.7		30.7	1,959,479.03	New Mexico
New York	2,578.8	15,432,510.33	3,199,755.00	215.7		215.7	9,187,745.10	3,217,960.00	157.4		157.4	9,955,121.04	New York
North Carolina	1,880.5	4,043,812.37	1,929,554.29	128.4	35.3	163.7	1,085,177.46	522,629.59	61.5	12.5	74.0	4,016,558.14	North Carolina
North Dakota	4,593.3	1,687,309.34	843,556.81	224.5	128.1	352.6	1,580,837.89	789,029.50	231.9	160.4	392.3	2,352,779.58	North Dakota
Ohio	2,530.1	12,199,792.49	3,802,480.18	175.3	12.1	187.4	646,893.50	188,078.24	2.7	1.2	3.9	4,570,245.34	Ohio
Oklahoma	1,971.8	4,812,195.60	2,281,290.20	160.6	64.1	224.7	1,395,638.24	752,911.18	95.8	7.3	102.9	2,064,244.28	Oklahoma
Oregon	1,235.7	5,741,166.10	3,333,929.64	214.8	75.3	290.1	1,220,929.60	730,765.91	51.4	15.3	66.7	1,212,223.43	Oregon
Pennsylvania	2,658.1	7,243,177.79	2,578,577.95	46.5		46.5	291,251.75	85,440.65	3.4		3.4	5,139,980.75	Pennsylvania
Rhode Island	209.5	1,720,027.14	564,901.84	25.4		25.4						634,535.18	Rhode Island
South Carolina	1,861.3	5,788,960.02	2,348,730.43	89.3	131.4	220.7	2,888,792.38	1,407,932.01	17.0	85.9	113.9	50,387.03	South Carolina
South Dakota	3,781.2	2,859,895.27	1,581,874.04	280.8	75.4	358.2	704,791.35	387,635.14	101.2	19.7	120.9	2,264,998.01	South Dakota
Tennessee	1,418.4	2,381,519.78	1,189,171.16	114.9		114.9	580,049.15	340,024.65	19.7		19.7	3,372,654.37	Tennessee
Texas	7,048.8	11,558,153.38	4,855,251.49	521.3	112.8	634.1	3,781,810.50	1,755,442.77	171.2	114.7	285.9	8,812,881.81	Texas
Utah	1,039.4	858,654.15	707,479.21	80.8	17.2	98.0	866,013.68	465,770.63	40.9	55.9	96.8	1,311,778.82	Utah
Vermont	303.8	203,087.09	62,014.73	2.7		2.7						622,221.98	Vermont
Virginia	1,512.7	3,915,802.31	1,889,100.80	165.0	11.7	157.7	1,707,098.68	732,740.10	63.3	18.8	82.1	1,611,090.20	Virginia
Washington	1,002.0	2,711,179.17	1,151,400.00	43.4	38.3	78.7	1,530,760.31	760,084.70	63.5		63.5	2,121,175.86	Washington
West Virginia	757.6	4,262,214.73	1,573,535.06	110.4	12.2	122.8	891,774.73	370,069.78	12.1	.3	12.4	860,581.98	West Virginia
Wisconsin	2,488.3	3,320,068.72	1,455,000.00	81.8	15.5	97.3	179,803.32	85,000.00	8.8		8.8	2,597,754.71	Wisconsin
Wyoming	1,771.2	2,573,533.30	1,715,157.11	161.0	94.0	255.0	779,057.68	511,451.27	22.1	109.6	131.8	973,977.75	Wyoming
Hawaii	41.2	1,080,386.08	472,874.00	30.7		30.7	43,765.70	21,272.25	1.1		1.1	1,831,497.33	Hawaii
TOTALS	89,572.1	223,508,255.79	96,571,264.28	5,857.3	1,855.0	8,812.3	80,911,823.88	35,450,099.41	2,566.5	1,162.1	3,728.8	130,223,875.73	TOTALS

[1] The term stage construction refers to additional work done on projects previously improved with Federal aid. In general, such additional work consists of the construction of a surface of higher type than was provided in the initial improvement.

U. S. GOVERNMENT PRINTING OFFICE. 1931

INDEX TO PUBLIC ROADS, VOLUME 11

A Page

Abrasion, *see* Rattler test—Wear.
Absorption tests and test results:
  brick 73, 156
  concrete aggregate 210
Accelerometers:
  calibrations, description, use 81–109, 111, 139–152
Acids in soils 78, 175
Accounting systems (North Carolina) 238–240
Administration, highway:
  road congress data 158–165, 168–170
  see also North Carolina.
Administration costs, motor vehicle taxes 44, 45
Admixtures:
  in concrete 209, 214, 216–220, 224, 225, 229–234
  in soils 78, 113, 120–122, 132, 188, 199, 202
Advertising rights as source of highway revenue (Italy) 162
Africa, road construction 160
Age-strength relations of concrete:
  curing study data 210–228, 230–234
Aggregate, mineral:
  character, variation in, as affecting stability of bituminous mixtures 207
  symposium, r., r. to p 185–187
  test methods 192
  types, sizes, application methods for bituminous roads 158, 168, 189–200, 202–208
  see also names of kinds of aggregate—*also* Asphalt concrete—Concrete aggregate—Grading—Spreading—Stain tests.
Agricultural lands:
  recovery following road construction 159
  taxation for highway purposes 160–164
Air hammer for soil sampling 40
Air in soils as affecting movement of water during freezing 116, 122, 123, 129
Air transport, coordination with highway transport 165, 170, 171
Airdromes, provision for 171
Airplane photographs, value 159
Alaskan ice heaves, soil blisters 63, 65–68, 79
Alignment chart (computation of truck requirements) 255, 256
Alignment of roads in undeveloped regions 159, 160, 168
Alkalinity of soils on metal culvert project 175
American Road Builders' Association:
  exhibition, announcements 35, 110, 111
American Society for Testing Materials:
  recent publications, announcement 235–236
  symposium, r., r. to p 185–187
Analyses, *see* Bituminous materials—Cement (test results)—Mechanical analyses—Metals, base—Soils—Stresses.
Anderson, Andrew P.:
  truck operation and production in concrete paving work, a 247–257, 260
Annuities, *see* Grants.
Apparatus, *see* names of kinds of apparatus—*also* names of tests and of processes in which apparatus is used.
Argentina, sand-clay road mixtures 159
"Armor coat" treatment of roads 199, 200
Asphalt:
  use, road congress data 153, 157, 158, 168, 201–208
  see also headings beginning Asphalt, Bituminous—*also* Cut-back products—Emulsions—Fluxes—Hunt Process—Oil asphalt—Oils—Paint coats—Seal coats.
Asphalt cement, use 194, 198–200, 202, 204–208, 259
Asphalt concrete paving work:
  high-speed production, a 258–260
  see also Bituminous roads and pavements (bituminous concrete pavements).
Asphalt content:
  of asphaltic paving mixtures 206, 207
  of bituminous materials (various) 192, 194, 198, 199, 201–203, 208
Asphalt heater, portable 188
Asphalt pavements, *see* Bituminous roads.
Asphalt plants, *see* Paving plants.
Assessments as source of highway revenue 161–164, 170
Austin-San Antonio Post Road, Texas:
  corrugated metal culverts, survey 173–181, 184
Automobiles, *see* headings beginning Motor.
Autostrades 163, 164, 169
Axles of motor vehicles:
  number as tax basis 46

B

Baldock, R. H. (collab.):
  road congress report 201–208
Ball compression test of brick, method 156
Barber, Alvin B.:
  road congress report 160–165
Barber asphalt emulsion, *see* Curcrete.
Barium sulphate, ice segregation in 119, 120, 125

Page
Base courses and foundation courses:
  concrete base courses 75, 76, 80, 153–157, 167
  crushed stone base courses 156, 197, 208
  for bituminous roads, bituminous treated roads 70, 75, 80, 153–155, 167, 196, 197, 202, 205–208, 258
  for brick pavements 154, 156, 157, 168
  macadam base courses 154
  thickness 154, 190, 202, 206
  see also Rubber pavements.
Batch production, *see* Concrete mix—Production studies.
Batchers, *see* Cement (handling).
Bates road test data, r 185
Bauman, E. W. (collab.):
  methods of curing concrete pavements, field investigation, a 209–234
Bazin's weir formula, use 32
Beam tests, *see* Concrete beams.
Bedding courses:
  for asphalt-block pavements 207
  for brick pavements 70, 156, 157
  see also Granular layers.
Belgium, brick pavements 156
Bending moments in concrete bridge slabs 1–23
Bentonite, *see* Colloid content.
Bernouilli and Navier theory, r 1
Bibliographies:
  concrete mixtures, design 187
  frost action 79
Binders, binding materials:
  bituminous binders—
    binder courses for hot-mixed, cold-mixed pavements 206, 208
    types and serviceability 74, 156–158, 168, 190, 192–208
  clay binders 79, 190
Bins, *see* Materials, highway (handling).
Bitumen:
  definitions 157, 201
  recent methods of use, road congress data 155, 157, 158, 168, 201–208
  see also headings beginning Asphalt, Bituminous.
Bituminous materials:
  analyses 70, 73, 192, 194
  as concrete curing agents 155, 188, 209–211, 213–220, 225–234
  research, recommendations 157, 158, 168
  types, quantities, application methods used 69–71, 73, 74, 157, 158, 168, 188–208
  see also names of bituminous materials—*also* headings beginning Bituminous—*also* Binders—Cut-back products—Distributors—Emulsions—Fillers—Fluxes—Hunt Process—Paint coats—Penetration—Priming coats—Sandstone—Seal coats.
Bituminous mats, *see* Bituminous treatment.
Bituminous paving mixtures:
  bituminous concrete mixtures, analyses (experimental road mixtures) 71
  cold-mix and cold-laid mixtures 208
  hot-mixed mixtures 206, 208
  mixing methods as affecting quality 158, 168, 207, 208
  patented (proprietary) mixtures, use 208
  use in foundations of brick pavements 156
  see also other headings beginning Bituminous—*also* Stability tests—Wearing courses.
Bituminous road mixer 189, 191
Bituminous roads and pavements:
  asphalt-block pavements—
    composition and construction methods 207
    impact test data 142–147, 149, 151
  asphaltic tops in resurfacing of roads in colonies 159
  bituminous concrete pavements
    bituminous surface treatment 70, 74, 80, 205
    composition and construction methods 206–208
    Connecticut Avenue experiments 69–74, 80
    high-speed production 258–260
  bituminous macadam pavements 69–75, 129, 130, 158, 205, 206
  construction—
    paving plant and construction details 207
    road congress data 157, 158, 160, 168, 201–208
  functions 154
  maintenance 70–74, 80, 158, 168, 201–208
  rock asphalt pavements 201, 207, 208
  sheet asphalt pavements—
    composition and construction methods 202, 206–208
    impact test data 142–148, 151
    surface treatment 205
  see also other headings beginning Bituminous—*also* Base courses—North Carolina—Slipperiness—Stability.

Page
Bituminous treatment, bituminous surface treatment of roads:
  Connecticut Avenue experiments 69–76, 80
  double-surface treatment 204
  methods in Western States 189–200, 208
  road congress data 155, 157, 158, 160, 168, 201–208
  single-surface treatment 204
  see also Retreatments.
"Black-base" foundations 206
"Black-waxy" soil 175
Blading methods and equipment 189–191, 199–205, 208
Blaney, H. F., and C. A. Taylor:
  soil sampling with compressed air unit, a 40
Blanket courses, *see* Bedding courses—Granular layers.
Bleeding of oil-mixed road surfaces 193, 197
Block pavements:
  asphalt-block pavements, *see* Bituminous roads.
  rubber block pavements 157, 168
  stone block pavements 151, 157
"Blow-sand" roads, oil treatment 198
Bond, *see* Binders—Bond tests.
Bond tests, announcement 172
Bonds, highway:
  bonding principles and practice 160–163, 165, 170
  issuance in undeveloped countries 160
  see also Finance.
Borrowing, *see* Bonds—Grants—Pay-as-you-go.
Bouyoucos, Geo. J., r., r. to p 51 (text and fn), 52 (text and fn), 62, 79
Brick, paving:
  specifications, tests, thickness, *etc.* 73, 156, 157, 168
Brick pavements:
  experiments, Connecticut Avenue 69–73, 75, 80
  impact test data 142–144, 146, 151, 152
  road congress data 154, 156, 157, 168, 205
  surface treatment 205
Bridges, highway:
  financing 38 (fn), 161, 162, 238, 239
  flooring, asphalt-block 207
  for pedestrian traffic at street intersections 172
  in new countries 159, 160, 168
  see also Concrete bridge slabs.
Briquet tests, cement:
  water for, measurement 182–183
Broken stone, *see* Crushed stone.
Brooming methods and equipment 191, 199, 200, 202, 204–207
Brown, Professor (McGill University), tests 62
Buchanan, James A. (J. A.):
  and G. P. St. Clair—
    calibrations of accelerometers, a 81–109, 111
  interrelated effects of load, speed, tires, and road roughness on motor truck impact, a 139–152
  motor truck impact as affected by rubber tread thickness of tires, a 133–138, 152
Budgets, highway:
  importance 164, 169, 170
Burettes, special, for cement briquet tests 182, 183
Burlap curing of concrete 187, 188, 209, 212, 215, 217–221, 224–226, 229–234
Busses, *see* Motor busses.
By-pass routes, recommendation 167

C

Calcium carbonate:
  use in subgrade treatment 78
  see also Soils (analyses).
Calcium chloride as concrete curing agent 155, 187, 188, 209, 213–220, 223–225, 229–234
Calibrations of accelerometers, a 81–109, 111
Caliche content of aggregate for oil-mix roads, significance 190, 192, 193
California Department of Public Works, research 40 (fn), 195 (fn)
Camps, convict, *see* Convict camps.
Cantilever beam tests of concrete:
  curing methods study data 210–214, 217–228, 230, 231, 234
Capillary moisture in soils:
  capillary action in subgrade, reduction 188
  movement 62–64, 68, 78, 79, 117, 122, 123, 129, 132
  see also Soil moisture—Tension.
Cement:
  briquet tests, measurement of water for 182–183
  handling 187, 249–253
  rapid-hardening cement, use 155, 167, 188
  test results (in concrete curing methods study) 210
  use, road congress data 153–155, 167, 184–188
  see also Bedding courses—Concrete—Fillers (cement grout)—Grout—Macadam pavements (cement-bound).
Cementing value of binder for use in mixed-in-place roads, limit 190

Page

- Chemical analyses, *see* Bituminous materials (analyses) — Cement (test results) — Metals, base — Soils (analyses).
- Chernozem soil 77
- Chips:
  - use in fine aggregate asphaltic concrete 206
  - *see also* Aggregate, mineral—Cover material.
- Cinders, use in subgrade 188
- Circuito (Spain) 163
- Classification of highways:
  - practices and recommendations 161, 162, 164, 169, 240, 242, 246
- Clay roads:
  - bituminous treatment 201, 202
  - *see also* Clays—Climatic conditions—Earth and gravel roads.
- Clays and clayey soils:
  - freezing, heaving 51, 52, 62–68, 79, 113–132
  - moisture absorption 201
  - *see also* Binders—Caliche—Clay roads—Granular layers—Sand-clay mixtures—Sand roads, sandy roads—Soils.
- Climatic conditions, weather conditions:
  - as affecting—
    - efficacy of concrete pavement curing method 209–234
    - serviceability of roads—
      - oil-mixed, clay, sand-clay roads 193–195, 197–199, 201–203
      - research, recommendations 158, 159, 168
    - soil formation 77–79
  - records of, suggested use 51–60
  - *see also* Cracks—Rainfall—Temperature—Temperature changes.
- Coefficients, *see* Bending moments—Calibrations—Flood water—Friction—Oil treatment of roads (oil requirements).
- Colloid content of soils:
  - as affecting permeability and heaving 78, 79, 113, 116, 120–123, 126, 132
  - colloidal particles, size 120
- Colonies and undeveloped regions:
  - road construction and finance 158–160, 164, 168, 169
- Colors of soils 78, 79
- Common carrier motor vehicles, *see* Motor vehicles.
- Compaction:
  - of bituminous road surfaces, methods and equipment 158, 168, 189, 190, 193, 195–197, 199–208, 258, 259
  - of soils, as affecting heaving 123–127
  - *see also* Finishing—Subgrades (construction).
- Comparator, special 86, 87
- Competition, *see* Motor trucks (contract hauling)—Transport, highway.
- Compression of air in freezing soils 116
- Compression tests:
  - of brick (ball compression test) 156
  - *see also* Compressive strength—Concrete beams, cores, and cylinders.
- Compressive strength:
  - of concrete—
    - as affected by—
      - aggregate 184–186
      - curing method 210–214, 217–228, 230, 231, 234
    - experimental road data 76
    - relation to transverse strength 186
  - of ice 62
- Compressive stresses, *see* Stresses (in concrete pavement slabs).
- Compressor in soil freezing apparatus 113, 114
- Compressor unit (air) for soil sampling 40
- Concessions (highway finance) 159
- Concrete:
  - high early strength concrete
    - making from ordinary cement, method 188
    - use 184, 188
  - quality, factors determining 155, 186, 187
  - *see also* headings beginning Concrete—*also* Admixtures—Bituminous roads and pavements (bituminous concrete . . .)—Cement—Compressive strength—Curing—Flexural strength—Proportioning—Rupture, moduli—Tensile strength—Transverse strength—Uniformity—Water-cement-ratio.
- Concrete aggregate:
  - characteristics as affecting strength of concrete, r., r. to p 155, 184–187
  - specifications, r 155, 185, 186
  - tests and test results, analyses 71, 73, 185, 210
  - type required for two-course pavement 154, 155, 167
  - weight corrections 187
  - *see also* names of kinds of aggregate—*also* Materials, highway—Proportioning (of concrete).
- Concrete base courses 75, 76, 80, 153–157, 167
- Concrete beams, cores, and cylinders:
  - test specimens—
    - data from curing methods study 210–214, 217–232, 234
    - practice, r 155, 185, 187
  - *see also* headings beginning Transverse—*also* Bernouilli—Compressive strength—Rupture, moduli.
- Concrete bridge slabs:
  - r. to p 1 (fn), 2 (fn), 4 (fn), 5 (fn), 9 (fn)
  - stresses due to wheel loads, computation, a 1–23

Page

- Concrete mix:
  - design 155, 167, 184, 186–188
  - water content, measurement 183, 184, 187
- Concrete mixers:
  - operating costs 248, 257
  - production data 187, 247–257, 260
  - types used 187, 188, 253
- Concrete mixing time 187, 188, 248, 260
- Concrete pavements:
  - bituminous wearing courses, bituminous surface treatment 69–76, 80, 205
  - construction materials, methods, and equipment—
    - r. to p 186, 187, 248 (fn)
    - road congress data 153–155, 167, 168, 184–188
    - truck operation and production 247–257, 260
  - construction without joints 167
  - costs, r 154, 155
  - design—
    - principles 153–155, 167, 184–187
    - research 172
    - *see also* Cross section design.
  - maintenance methods and costs, untreated and surface treated roads 70–72, 75, 76, 80, 154, 155, 167, 168, 184, 188
  - mileage, United States, 1928 154, 184
  - movement 172, 185
  - single-course pavements, suitability for rubber tired traffic 154, 155, 167
  - two-course pavements 153–155, 167
  - *see also* other headings beginning Concrete—*also* Bituminous roads and pavements (bituminous concrete . . .)—Concrete—Cracks—Curing—Edge thickening—Flexural strength—Friction—Impact—Joints — North Carolina — Oil cement concrete—Paving plants—Roughness—Stresses—Subgrades—Thickness—Wear.
- Concrete reinforcement:
  - base course reinforcement (brick pavements) 156
  - pavement reinforcement—
    - embedment, length, research 172
    - road congress data 153–155, 184–186
    - specifications 186
- Congresses, road, *see* Road congresses.
- Connecticut, State of:
  - report for (motor vehicle taxation) 41–50
- Connecticut Avenue experimental road, Montgomery County, Md.:
  - progress report 69–76, 80
- Consistency of concrete, recommendation 187
- Contract hauling, motor truck 165, 171, 247, 257, 260
- Contraction, *see* Cracks—Shrinkage—Stresses.
- Contractors, concrete road construction:
  - penalizing 155, 187
- Convict camps, North Carolina 239, 243–246
- Corrosion resistance of corrugated metal culverts 175, 177–181, 184
- Costs (general statements and cost data), *see* Administration costs—Concrete mixers (operating costs)—Concrete pavements (costs—maintenance)—Drinking fountains—Federal-aid—Finance—Gasoline tax—Macadam pavements (maintenance . . .)—Machine methods—Maintenance—Motor vehicles (costs to State)—Oil treatment—Operating costs—Paving plants—Registration fees—Surface loss—Time losses—Transportation costs.
- County road and finance survey, North Carolina 237–246
- Cover material:
  - types, application methods, *etc* 157, 158, 168, 188, 191, 196, 199, 208
- Cracks:
  - in concrete pavements—
    - formation as affected by—
      - curing method 155, 187, 188, 209–211, 215–228, 230, 232–234.
      - weather conditions (temperature, rainfall) 185, 216–228, 230, 232–234
        - *see also* Freezing.
      - width of slab strip 185
    - research, recommendation 167
    - *see also* Concrete reinforcement—Fillers—Joints.
  - in frozen soils 65–67, 117, 120–129
  - in pavements (general)—
    - formation as affected by subgrade conditions 76, 128–132, 154, 185
- Critz, Paul F., and J. H. Eldridge:
  - Connecticut Avenue experimental road, a 69–76, 80
- Cross section design:
  - of city streets 166, 167
  - of concrete pavements 154, 184, 185, 187, 209
  - *see also* Crowns, reversed.
- Crowns, reversed:
  - use (in Siam) 159
- Crum, Roy W. (R. W.):
  - r. to p 186, 187
  - rating schedule, metal culverts 177
  - road congress reports 157–158, 186–187
- Crushed stone:
  - sizes used, r 71, 73, 190, 199, 200, 202–208, 259
  - *see also* Aggregate, mineral—Base courses—Concrete aggregate—Cover material—Granular material.

Page

- Crushed stone roads, *see* Bituminous treatment—Macadam pavements—Surface loss.
- Cuba:
  - road congress report, r 159
- Culverts:
  - corrugated metal culverts, survey 173–181, 184
  - movement upward toward surface 131
- Curbs, lip, in concrete pavement design:
  - research 172
- Curcrete as concrete pavement curing agent 209, 213, 216–220, 225–227, 229–234
- Curing:
  - of cold-patch bituminous mixtures 208
  - of concrete pavements—
    - methods—
      - road congress data 155, 167, 184, 187, 188
      - Tennessee study 209–234
    - *see also* Curing periods—Moisture loss.
- Curing periods, concrete pavement:
  - r 187, 188, 209, 212, 215, 217–222, 229–234
  - short periods under low temperature conditions as affecting pavement crack formation 220, 221
- Curves, highway, in undeveloped regions 159, 160, 169
- Cushioning properties of motor truck tires:
  - as affected by rubber tread thickness 133–138, 152
- Cut-back products:
  - as concrete pavement curing agents 209–211, 213–220, 225–227, 229–234
  - cut-back asphalts, definition 201
  - test results 194
  - use in bituminous treatment of roads 157, 158, 190, 193–195, 197–199, 201, 203–205, 208
  - *see also* Connecticut Avenue.
- Cuts, construction 79, 169

D

- Davis, T. M.:
  - drinking fountains along Oregon highways, a 25–29
- Dean, A. W. (collab.):
  - road congress report 201–208
- Deflections and deformations, *see* Concrete bridge slabs—Impact—Static load-deflection data.
- Denmark, road construction and finance 161
- Densities and volumes of freezing water 61, 79
- Desert roads 160, 169
- Design, highway, and construction methods:
  - road congress data 153–160, 166–169, 171, 172, 184–187, 201–208
  - *see also* names of types of highways and names of features of design.
- Disking, harrowing, methods and equipment, r 189, 196, 202, 203
- Displacement-time recording apparatus and records, impact test data 81–88, 90–94, 98, 103, 104, 111
- Distillates, types used 157, 190, 193–195, 197–199, 201, 204, 205, 208
- Distillation test results, bituminous materials 70, 73
- Distributors, pressure:
  - installation for emulsified asphalt application 158, 204
  - use, r 158, 189, 191, 196, 201–207
- Dokuchaev school of soil science 77
- Dowel action of concrete pavement reinforcement, dowel bars 155, 172, 185
- Drainage:
  - drainage system of experimental road 72, 74–76
  - natural drainage conditions as affecting choice of type and location of roads 159, 167
  - of subgrades, r 78, 154, 156, 185
  - *see also* Culverts—Freezing (of soils).
- Drilling of concrete cores 155, 187, 211, 232
- Drinking fountains along Oregon highways, a 25–29
- Drop reaction, *see* Impact.
- Drop test of brick, r 156
- Dryer temperature (asphalt concrete paving work) 259
- Dust:
  - handling in asphalt concrete paving work 259
  - palliation by oil treatment 192, 201, 202, 204
  - removal before bituminous treatment of roads 202, 204, 205

E

- Earth and gravel roads:
  - bituminous treatment 157, 158, 160, 168, 189–205, 208
  - in undeveloped regions 158–160, 168, 169
  - moist surfaces 203
  - research, recommendation 169
  - surface loss 197
- Earth curing of concrete (wet earth, poor earth) 155, 187, 188, 209, 214, 216–222, 224, 225, 228–234
- Edge reactions in concrete bridge slab under load 1–23
- Edge thickening:
  - of bituminous surfaces 190, 258
  - of concrete pavements—
    - practice, formulas 154, 155, 167, 185
    - research 172
  - of transverse joints in concrete pavements, research 172
- Edwards, R. W., and N. L. James:
  - high-speed production on asphalt concrete paving work, a 258–260
- Elasticity, theory of:
  - application in concrete bridge slab analysis 1–23
- Eldridge, J. H. (collab.):
  - Connecticut Avenue experimental road, a 69–76, 80

Page
Electric motor vebicles, taxation 43, 46, 49
Elevated streets 167, 172
Embankments:
erosion, reduction by reversed crowns 159
flow of flood water over 30-34
Emulsions, bituminous:
as concrete pavement curing agents 209, 213, 216-220, 225-227, 229-234
emulsification methods. Oregon 200, 204
use 69-71, 74, 158, 168, 193, 196, 198-200, 204-206, 208
Engineering, highway, and soil science:
points of contact, a 77-79
England, *see* Great Britain.
Equipment, *see* names of kinds of equipment—*also* names of processes and of tests in which equipment is used.
Erosion, *see* Embankments.
Expansion:
of air in soil 116
of concrete, *see* Cracks—Joints—Stresses.
of water during cooling and freezing 61, 62, 113, 115-118
see also Swelling.
Expenditures, highway, *see* Federal-aid—Finance—Gasoline tax—Registration fees.

F

Federal-aid road construction:
current status rear covers of Nos. 1-6, 8-12
see also Production studies.
Fees, *see* Registration fees—Taxes.
Feldspar content of soils as affecting ice segregation 123
Fillers:
bituminous fillers, use 155, 157, 188
cement grout fillers, use 157, 207
for block pavements (stone, rubber, asphalt) 157, 168, 207
for brick pavements 156, 157
for concrete pavement cracks and joints 155, 168, 188
mineral fillers in bituminous paving mixtures 202, 206, 207
Fills, earth 68, 79, 169
Finance, highway:
North Carolina county road and finance survey 237-246
road congress data 158-165, 169, 170
see also Bonds—Federal-aid—Grants—Revenues—Taxes.
Fineness of cement, of aggregate:
test results 210
Fines:
as source of highway revenue (Italy) 162
see also Registration fees.
Finishing methods and equipment:
in bituminous road construction 158, 201-208, 258, 259
in concrete road construction and maintenance 187, 188
in subgrade construction 185
see also Time losses.
Finland, road development 159
Fleming, E. M., road congress report 187-188
Flexural strength:
of bituminous, brick, and block surfaces, r 154
of concrete pavements—
as affected by—
aggregate 184-187
cement quality 186
curing method 209-214, 217-228, 230-232, 234
balancing in slab thickness 185
see also headings beginning Transverse—*also* Bending moments—Concrete beams—Concrete bridge slabs.
Flood water:
flow over embankments, a 30-34
Fluxes:
fluxed native asphalt, use in experimental road 69-71, 73, 76, 80
fluxing media, fluxing processes, r 193-195, 198, 201, 208
see also Cut-back products.
Form installation, forms 187, 190, 202, 248, 258
Foundations, *see* Base courses.
Fountains, drinking, *see* Drinking fountains.
Fourier series, use and r. to p 5 (fn)
France and French colonies, roads 156, 160, 161
Freeman, P. J., road congress report 156-157
Freezing:
of soils—
analysis of data on frost occurrence, method, a 51-60
critical depth, critical temperatures 51-56, 59, 60
damage to pavements—
freezing and thawing as factors in, a 113-132
reduction 68, 79, 113, 130-132
frost and ice phenomena, a 61-68, 79
r. to p 51 (fn), 79, 113 (fn), 115 (fn), 117 (fn), 127 (fn)
see also Air in soils—Climatic conditions—Cracks (in concrete pavements)—Temperature.
Freight, transportation by motor truck 165, 170
Frickstad, *see* McKesson and Frickstad.
Friction, subgrade:
coefficient 185
research 172
Frictional resistance of freezing soils 114, 118, 120, 125, 126, 131
Frost boils, *see* Freezing.

Page
Fuel oils, use in "armor-coat" treatment 199
Funds, highway, *see* Federal-aid—Finance—Revenues—Taxes.

G

Garaging of road vehicles 153, 167, 172
Gasoline:
as fluxing medium for bituminous materials 194
gallons purchased per vehicle, gallons taxed (statistics) 36, 39, rear cover of No. 9
Gasoline tax:
revenues and disposition of, rates, *etc.*, for 1929 36, 39
1st half of 1930 rear cover of No. 9
see also Revenues—Taxes.
Germany, highways and highway finance 156, 159, 161, 162
Gley 78
Grade crossings, railroad:
elimination, protection 159, 161
Grades, highway:
grade lines, fixing with regard to soil horizons 79
grade separation at intersections 167, 172
in undeveloped regions 159, 160, 169
Grading:
highway grading—
mechanical grading in desert regions 169
see also Blading.
of aggregate for bituminous road surfaces 168, 190, 192, 195, 203, 206-208
Granite:
as soil former 77
see also Aggregate, mineral—Block pavements—Concrete aggregate.
Grants, subsidies, loans:
for road construction 159, 161-164, 169, 170
to highway transportation enterprises 166, 171
see also Bonds—Finance.
Granular layers, blanket courses 113, 132, 156, 185, 202, 206
see also Bedding courses.
Granular material:
as admixture in soils 132, 188, 199
as fine aggregate in mixed-in-place treatment of roads 190
see also Aggregate, mineral—Granular layers—Gravel—Sands.
Gravel:
as base course for oil-mixed road surfaces, cost 197
see also headings beginning Granular—*also* Aggregate, mineral—Cover material—Gravel concrete—Macadam pavements.
Gravel concrete experimental road:
surface treatment 69-76, 80
Gravel roads, *see* Earth and gravel roads.
Gravity, specific, *see* Specific gravity.
Great Britain:
highways and highway finance 156, 157, 159, 162
Ground water, *see* Freezing (of soils)—Water table.
Grout:
for brick experimental pavement 70
for cement-bound macadam pavements 153
see also Fillers.
Gumbo, heaving 113, 116, 122
Gypsum as subgrade admixture 78

H

Hardening rate of road oils 194
Hardness, *see* Hardening rate—Wear.
Hardpan 68, 78
Harmonic motion machine, impact test data 81-101, 111
Harrowing, *see* Disking.
Haul lengths, hauling operations:
in paving work, production study data 247-257, 259, 260
see also Trip mileages.
Hay, wet, as concrete pavement curing agent 187
Headley asphalt emulsion:
as concrete pavement curing agent 209-211, 213, 216-220, 226-234
Heat radiation, heat conduction:
as affecting growth of ice crystals 62, 63, 113, 117, 118, 123-125, 128, 129
Heaving of soils, *see* Freezing.
Henderson, G. H. (collab.):
road congress report 201-208
Hogentogler, C. A., r. to p 185, 186
Holland, highways and highway finance 156, 159, 162
Horsepower, motor vehicle, as tax basis 41, 49, 50
Hubbard—Field stability test results, bituminous sand mixtures 193, 194
Hubbard, Prevost (collab.):
road congress report 201-208
Humus content of soils 78, 123
Hunt Process:
use in concrete pavement curing 209-211, 213, 215-220, 225-227, 229-234
Hydraulic formulas (flood water computations) 32-34

I

Ice phenomena, *see* Freezing.
Impact, motor truck:
accelerations caused by, methods of measuring; calibrations of accelerometers, a 81-109, 111
as affected by rubber tread thickness of tires, a 133-138, 152
interrelated effects of load, speed, tires, and road roughness, a 139-152
see also Stresses (impact stresses).

Page
Impact testing machine, motor truck 81-85
Improvement, highway:
investment character 164, 165, 170
see also Agricultural land—Finance—Operating costs—Planning—Roadside plantings—Taxes.
Indexes, announcements 236
Indo-China, road construction 160
Industrial railroads, use 187
Industries using highways:
levies on (Italy) 162
Inspection, *see* Supervision.
Insulating effect of dry and of wet earth on new concrete:
research, recommendation 229
Intersections, highway:
designs, traffic regulations 159, 161, 167, 171, 172
Ireland, highways and highway finance 162
Iron as soil constituent 77, 78
Iron culverts, *see* Culverts.
Italy, highways and highway finance 159, 162, 163

J

Jack, soil tube, r. to p 40 (fn)
Jackson, F. H., and E. W. Bauman:
methods of curing concrete pavements, field investigation, a 209-234
James, E. W., road congress report 158-160
James, N. L. (collab.):
high-speed production on asphalt concrete paving work, a 258-260
Joffe, J. S.:
points of contact between soil science and highway engineering, a 77-79
Johns Hopkins University:
impact tests, r., r. to p 152 (text and fn)
Johnson, A. N., road congress report 185-186
Joining material for rubber paving:
research, recommendation 168
Joints:
in asphalt-block pavements 207
in concrete pavements 155, 167, 168, 172, 181, 185, 188, 209, 227, 248
see also Fillers.

K

Kadox, ice segregation in 119, 120, 125
Kamerun, road construction 160
Kaolin content of soils as affecting ice segregation 123, 131
Kellermann, W. F., r. to p 186, 187
Kelley, E. F.:
r. to p 1 (fn), 9 (fn)
survey of corrugated metal culverts, a 173-181, 184
Kerosene, use:
as fluxing medium for bituminous materials 190, 194, 195, 197-199
in cold-laid bituminous paving mixtures 208

L

Labor, *see* Convict camps—Poll-tax labor—Production studies.
Lamp, neon, used in calibration tests 89, 90
Landslides, *see* Soil flow.
Lanes, *see* Traffic lanes.
Laterite soils 77
Leaching process in soils 78
Leaf-mold in soils 78
License fees, licenses, *see* Registration fees.
Lime content of soils 79, 175
Limestone:
as aggregate constituent, effect upon oil requirements of mixed-in-place road surfaces 195
as block pavement material 157
as soil former 77, 175
bituminous limestone, use 207, 208
see also headings beginning Limestone—*also* Aggregate, mineral—Caliche—Concrete aggregate.
Limestone concrete experimental road:
surface treatment 69-76, 80
Limestone dust as mineral filler in hot-mixed pavements 206
Lineal shrinkage, limiting value:
for binder for mixed-in-place road surfaces 190
Lithopone, ice segregation in 119, 120
Load, loads:
of motor vehicles, limitation 159, 169
surface load as affecting ice segregation in soil 113, 125, 126
transference across joints in concrete pavements, research 172
see also Ball compression test—Edge thickening—Impact—Load capacities—Loading operations—Pressure—Static load-deflection data—Stresses.
Load capacities, load resistance:
of roads and pavements—
miscellaneous r 153-155, 157, 167, 168, 201
relation to thickness 154, 155, 167, 172, 185, 201
see also Tires—Traffic capacities.
Loading operations:
offstreet loading and unloading, provision for 172
see also Materials, highway (handling).
Loans, *see* Bonds—Grants.
Local roads, *see* Classification—Federal-aid—Finance—North Carolina—Revenues—Undeveloped regions.

M Page

- Macadam pavements:
  - bituminous macadam pavements 69–75, 129, 130, 158, 205, 206
  - bituminous treatment, bituminous surface treatment 69–75, 157, 158, 189, 198–200, 204, 205
  - cement-bound macadam pavements 153–155, 167
  - failure through heaving of subgrade and shoulders 129, 130
  - impact test pavements 142
  - in undeveloped countries 159, 160
  - maintenance and maintenance costs 70–76, 197
  - *see also* Base courses.
- Machine methods of road construction:
  - economy of 155, 159, 167, 187, 204
  - *see also* names of machines and names of processes in road construction.
- Maintenance methods and costs, highway:
  - comparative maintenance costs of—
    - bituminous macadam and surface treated sections of experimental road 70–76
    - oil treated and untreated roads 197
  - patrol maintenance of sand-clay roads 203
  - relation of maintenance costs to volume of traffic—
    - research, recommendations 158, 168
  - road congress data 153–155, 158, 160–171, 184, 188, 201–208
  - *see also* names of types of highways *also* Finance.
- Maps:
  - for use in study of ground freezing 57, 60
- Marking, highway, *see* Traffic lanes—Traffic signs.
- Marvin, Charles F., Jr.:
  - technical basis for apportioning motor vehicle taxes, a 41–50
- Materials, highway:
  - handling methods and equipment 184, 187, 247–260
  - in undeveloped countries—
    - research, value 159
  - *see also* names of materials *also* Replacements.
- M'Clintock, Dr. Miller:
  - road congress report 166–167
- McKesson and Frickstad formula (oil requirements of oil-mix roads) 192, 195 (text and fn)
- Measurement of water for cement briquet tests, a 182–183
- Measuring devices for concrete mixers 183, 184, 187
- Mechanical analyses:
  - of aggregates 71, 73, 192, 210
  - *see also* Soils (analyses).
- Metals, base, for corrugated pipe culverts:
  - culvert survey data 173–181, 184
- Michigan highways:
  - traffic survey, announcement 172
- Mileages:
  - of motor vehicles during taxable period as tax factor 41, 42, 45
  - *see also* Concrete pavements—Federal-aid—Haul lengths—North Carolina—Trip mileages.
- Military roads 158 160, 164, 169
- Mineral constituents of soil:
  - as affecting ice segregation 123
  - *see also* Soil science—Soils (analyses).
- Mix, mixing methods, *see* Asphalt concrete paving work—Bituminous paving mixtures—Concrete mix—Concrete mixing time—Mixed-in-place road surfaces.
- Mixed-in-place road surfaces:
  - construction methods and equipment 189–200, 202–205, 208
  - maintenance methods and costs 190, 196, 197, 203
- Mixers:
  - electric mixer for use in aggregate test 192
  - *see also* Bituminous road mixer—Concrete mixers—Paving plants.
- Moisture:
  - effects upon bituminous treated roads, resistance to 193–199, 201, 202, 207
  - *see also* headings beginning Moisture, Water — *also* Absorption — Climatic conditions — Concrete aggregate (tests)—Curing—Drainage—Freezing—Rainfall —Soil moisture.
- Moisture changes as affecting:
  - crack formation in concrete pavements 220, 232
  - movement of concrete slab in subgrade, research 172
- Moisture content:
  - of aggregate—
    - as affecting oil requirements of mixed-in-place roads 196
    - determination 187
  - *see also* Water content.
- Moisture equivalent, limiting value:
  - for binder for mixed-in-place roads 190
- Moisture loss from concrete during curing:
  - avoidance by wetting of subgrades 154
- Moments (bending, twisting) in concrete bridge slabs 1–23
- Moorefield, Charles H. (collab.):
  - road congress report 201–208
- Morocco, road development 160
- Mortar-void method of design of concrete mixtures 184, 186
- Motor busses 45, 46, 49, 165, 170, 171
  - *see also* Motor transport—Registration fees—Registrations.
- Motor cycles:
  - registrations, registration receipts (1929) 37, 38
- Motor transport:
  - as influencing highway design, planning, and finance 158–165, 168–171
  - correlation with other means of transport 153, 159, 163–167, 169–171
  - *see also* other headings beginning Motor— *also* headings beginning Traffic.
- Motor trucks:
  - contract hauling 165, 171, 247, 257, 260
  - operation in paving work, truck types used 187, 188, 247–260
  - truck requirements, concrete paving jobs 254–256
  - use in impact tests, test truck equipment 82, 83, 95, 134, 139–152
  - *see also* Freight — Haul lengths — Motor vehicles —Registration fees—Registrations—Taxes—Trip mileages.
- Motor vehicles:
  - common-carrier motor vehicles 163, 165, 170, 171
    - *see also* Motor busses—Motor transport.
  - costs to State, computation method 45, 46
  - speed—
    - as affecting impact 106–109, 111, 134 152
    - regulation 149, 169
    - relation to rated power 49
  - *see also* other headings beginning Motor— *also* headings beginning Traffic— *also* Registrations— Revenues—Taxes—Tires.
- Muck, permeability and heaving 113, 116, 122, 123
- Mud boils 65, 66, 68
- Mullis, Ira B.:
  - frost and ice phenomena, a 61–68, 79
- "Multiple lift" treatment of roads 200

N

- Nagler, Floyd A. (collab.):
  - flow of water over railway and highway embankments, a 30–34
- Naphtha as fluxing medium, bituminous materials 198
- Navier (and Bernouilli) theory, r 1
- Nebraska Department of Public Works:
  - oil treatment experiments 198
- North Carolina county road and finance survey 237–246

O

- Oil asphalt, use 69–73, 195
- Oil cement concrete experimental road data 69 76, 80
- Oil-mixed roads:
  - bituminous treatment 196–200, 203
  - *see also* Oil treatment.
- Oil taxes as source of highway revenue 159, 163
- Oil treatment of roads:
  - cost data 197, 198
  - methods and equipment 160, 189–204, 208
  - oil requirements 192, 195–198, 203
  - *see also* Bituminous treatment.
- Oils, petroleums, for road use:
  - specifications and test results (Western States) 192–194
  - types (various) with results obtained from use 69–74, 189, 190, 192–204, 208
- Older, Clifford, road congress report 184–185
- One-way movement of traffic, recommendation 171
- Operating costs:
  - of vehicles —
    - as affected by highway improvement 163, 164, 169
    - research, recommendations 158, 168
  - *see also* Concrete mixers—Paving plants.
- Oregon highways:
  - drinking fountains, a 25–29
- Organic content:
  - of fine aggregate, test for detecting 185
  - of soils —
    - as affecting—
      - corrosion of metal culverts 175, 180
      - ice segregation 123
    - *see also* Soil science.

P

- Paint coats in pavement maintenance 74, 188
- Painting of concrete test beams in curing methods study 211, 217
- Paper, *see* Sisalkraft paper—Tar paper.
- Parking, vehicle:
  - parking spaces at drinking fountains 25
  - restriction 153, 166, 167, 171
- Passing places on roads in undeveloped countries 159
- Patrol maintenance, *see* Maintenance.
- Pauls, J. T.:
  - bituminous treatments used on Western roads, a 189–200, 208
- Paving plants, plant mixing:
  - for bituminous road work 188, 189, 191, 202, 204, 207, 208, 258–260
  - operation, operating costs 248, 250–252
- Pay-as-you-go plan in highway finance 163, 165, 170
- Peat:
  - shrinkage 79
  - supercooling, duration period 51
- Pedestrian traffic control 166, 167, 171, 172
- Penetration, rainfall:
  - studies, r 40
- Penetration methods, *see* Bituminous treatment.
- Penetration of bituminous materials:
  - as affecting stability of bituminous paving mixture 207
  - miscellaneous, r 70, 73, 192, 194, 195, 197, 201, 202
- Permeability and porosity of soils: Page
  - as affecting heaving 63, 79, 113, 116, 120, 122–126, 130, 132
    - *see also* Freezing.
- Permits, motor vehicle, *see* Registration fees.
- Petroleums, petroleum products, *see* Binders—Bitumen—Bituminous materials—Oils.
- Photographs, airplane, value 159
- Piston displacement of motor vehicles as tax basis 41, 48 50
- Placing of concrete, *see* Concrete pavements (construction)—Replacements.
- Planning and financing of highways:
  - road congress data 158–172
  - *see also* Finance—Taxes.
- Plant mixing, *see* Paving plants.
- Plantings, roadside, *see* Roadside.
- Poisson's ratio, *see* Stresses (in concrete bridge slabs).
- Poland, road construction 156, 159
- Poll-tax labor (Poland) 159
- Poll taxes as source of highway revenue (Spain) 163
- Ponding as concrete pavement curing method 155, 187, 209–211, 214, 216–220, 228, 229, 231–234
- Porosity:
  - of concrete aggregate as affecting strength of concrete 186
  - of oil-mixed road surfaces 197
  - *see also* Permeability.
- Powers, J. W., aggregate test 192
- Pressure:
  - atmospheric pressure —
    - as affecting freezing point of water 115, 118, 126
    - for water measuring tanks of concrete mixers 187
  - exerted by freezing water, pressures developed during freezing of soils 61–68, 79, 113–132
  - exerted by liquids that freeze with decrease in volume 116, 127
  - external pressure as affecting freezing point of water and growth of ice crystals 61, 79, 113–118, 124–131
  - *see also* Impact—Load.
- Pressure distributors, *see* Distributors.
- Priming coats, priming materials, for bituminous road work 74, 199, 200, 202, 204, 208
- Production studies:
  - of asphalt concrete paving work 258–260
  - of concrete paving work 247–257, 260
- Programs, highway, *see* Improvement—Planning.
- Property taxes, *see* Taxes (general).
- Proportioning:
  - of bituminous mixtures 158, 168, 189, 203
  - of concrete, methods 155, 167, 184–187, 210
  - of concrete reinforcement 155, 185

Q

- Quartz content of soils as affecting ice segregation 118–120, 12

R

- Railroads, *see* Embankments—Grade crossings—Transport, highway.
- Rainfall:
  - data and significance in various studies 55, 57, 77, 174, 209–234, 247, 258, 260
  - penetration, investigations, r 40
  - *see also* Climatic conditions—Drainage—Flood water—Freezing—Moisture.
- Rating schedule for corrugated metal culverts 173, 177, 181
- Rattler test and test results (paving brick) 73, 156
- Raveling of bituminous road surfaces 193–195, 197, 198, 200, 203
- Reaction wheel tests in accelerometer investigation 81–109, 111
- Reactions in concrete bridge slabs under load 2, 22, 23
- Reagel, F. V.:
  - method for measurement of water for cement briquet test, a 182
- Reconnaissance surveys:
  - in undeveloped regions, r 159
- Recreational value of roads, r 172
- Refrigerator, type used in soil tests 113 115
- Refunds of gasoline tax and registration fee receipts 38 (fn), 39 (fn), rear cover of No. 9
- Registration fees, licenses, *etc.* (motor vehicle):
  - highway revenue from 36, 38, 160–165, 170
  - *see also* Registrations—Taxes.
- Registrations (motor vehicle):
  - United States, 1929 36, 37
- Regulation, traffic, *see* Traffic regulation.
- Relief funds, unemployed, as source of highway revenue (Germany) 162
- Repair, pavement, *see* Maintenance—Replacements.
- Replacements:
  - concrete pavement replacements 155, 188
  - material replacement costs, untreated and oil-treated roads 197
- Research, highway:
  - program of Bureau of Public Roads 172
  - recommendations 59, 113, 132, 154, 155, 157-159, 168, 169, 184, 186
  - *see also* names of highway materials and names of processes in road construction.
- Residue:
  - test results for various materials 70, 73, 192, 194, 210
- Retread treatment of roads 204, 205

Page

- Retreatments of bituminous treated roads 74, 196, 197, 199–206
- Revenues, highway:
  - motor vehicle revenues (including fuel tax receipts)—
    - increase in United States 160, 163
    - statistics and miscellaneous r 36, 38, 39, 161–166, 170; rear cover of No. 9
    - use for other than highway purposes 36, 38, 39, 161–164, 170, 245, 246; rear cover of No. 9
  - sources (in foreign countries and in United States) 36, 38, 39, 159–166, 169, 170; rear cover of No. 9
  - *see also* Bonds—Federal-aid—Finance—Grants—Taxes.
- Riding qualities of roads 200, 203, 207
- Rights of way 159, 160, 165, 168
- Road congresses:
  - permanent international association (membership, *etc.*) 35 (text and fn)
  - sixth international congress—
    - announcements and reports 35, 36, 110, 111, 132, 153–172, 184–188, 201–208
  - third international congress—
    - conclusions on highway finance 160, 161
- Road signs, *see* Traffic signs.
- Roads and road systems, in foreign countries 156, 163, 168, 169
- Roadside plantings in undeveloped countries 160
- Rock asphalt, rock asphalt pavements 201, 207, 208
- Rolling methods and equipment 157, 168, 185, 190, 193, 199, 200, 202, 204–208, 258, 259
- Rotary movement of traffic 159, 167, 171
- Roughness, pavement:
  - as affecting truck wheel impact 81, 87, 106–109, 133–152
  - maximum permissible, concrete pavements 187
  - *see also* Roughness indicator—Smoothness.
- Roughness indicator, relative:
  - r., r. to p 142 (text and fn), 143, 151, 152
- Rubber, tread, *see* Tires.
- Rubber pavements, rubber paving blocks 157, 168
- Rupture, moduli of, for concrete 154, 185, 211–214, 217–228, 230, 231, 234
- Rural roads, local roads, *see* Classification—Federal-aid—Finance—North Carolina—Revenues—Undeveloped regions.
- Rust, *see* Rating schedule.
- Ruts, *see* Stability.

S

- Safety:
  - factor of, *see* Edge thickening.
  - highway safety, *see* Traffic regulation.
- Sahara, Desert of:
  - road construction projects 160
- Salts:
  - hygroscopic salts, soils containing—
    - research, recommendation 169
  - *see also* names of salts—*also* Soils (analyses).
- Salvaging road construction:
  - in undeveloped regions 158–160
- Sampling, soil, *see* Soil sampling.
- San Antonio Post Road, *see* Culverts.
- Sand-clay mixtures:
  - ice segregation in 120, 122
  - research 159, 169
  - *see also* Sand roads.
- Sand roads, sandy roads:
  - bituminous treatment 189, 198, 199, 201–203
  - "blow-sand" roads 198
  - sand-asphalt roads 202
  - sand-clay roads 79, 201–203
  - *see also* North Carolina—Sand-clay mixtures.
- Sand subgrades for brick pavements 156
- Sands and sandy soils:
  - as subgrade admixtures, effect upon heaving 113, 132
  - freezing, heaving 51, 52, 62–64, 66, 67, 113, 116, 120, 122, 128, 129, 132
  - profile characteristics 78, 79
  - *see also* headings beginning Granular, Sand, Soil—*also* Aggregate, mineral—Bedding courses—Concrete aggregate—Cover material—Fillers—Grout—Hubbard-Field Sandstone—Supercooling.
- Sandstone:
  - as block pavement material 157
  - bituminous sandstone, composition and use 207, 208
- Scaling of concrete test pavement 224
- Scarifying, practice 189, 190, 196, 199, 203–205
- Screeding of concrete pavements 187, 188
- Screenings, stone, *see* Bedding courses—Cover material.
- Seal coats, materials and application methods 188, 203, 205, 206, 208
- Selection of highway type:
  - principles 154, 159, 160
- Settlement of foundations as cause of pavement failures 72–74, 76, 80, 130
- Setts (brick or stone), use 159
- Shale:
  - as brick constituent 156
  - as soil former 77, 78
- Shear:
  - in concrete bridge slabs under load 1–7, 20–23
  - shear action caused by concrete pavement reinforcement 155, 185
- Sheet asphalt, *see* Bituminous roads.
- Sheets, Frank T.:
  - road congress reports 153–155, 184
- Shoulders, pavement:
  - for bituminous macadam pavements 206
  - heaving 129
    - *see also* Freezing.
- Shrinkage:
  - of concrete, *see* Cracks—Joints—Stresses.
  - of soils 67, 79, 123
    - *see also* Freezing.
- Siam, highways and highway finance 159, 163
- Siberian subsoils, r. to p 79
- Sieve analyses, *see* Mechanical analyses.
- Signs, *see* Traffic signs and signals.
- Silica in soils 78
- Silts:
  - heaving 63, 68, 125
    - *see also* Freezing—Permeability.
  - percentage in sand-clay road mixture 202
- Sisalkraft paper as concrete pavement curing agent 209–212, 215, 217, 218, 220, 222, 229–234
- Slag, slag roads, *see* Aggregate, mineral—Base courses—Bedding courses—Bituminous treatment—Cover material.
- Slipperiness, pavement 157, 158, 168, 207
- Smoothness:
  - of base courses for brick pavements 156
  - of brick pavements 157
  - of concrete pavements 154, 155, 184, 187
  - *see also* Roughness.
- Snow as affecting freezing of soils 51, 130, 131
- Sodium hydroxide test, concrete aggregate 185
- Sodium silicate:
  - as admixture in soils, effects upon freezing point 132
  - as concrete pavement curing agent 155, 187, 188, 209–211, 213, 215, 217–220, 222–224, 229–234
- Soil flow 68, 79, 130
- Soil horizons 77–79
- Soil moisture:
  - studies, r 40
  - *see also* Capillary moisture—Freezing (of soils)—Moisture—Moisture equivalent—Soil science—Water content—Water table.
- Soil profiles 77–79
- Soil sampling with compressed air unit 40
- Soil science and highway engineering:
  - points of contact, a 77–79
- Soil temperatures, *see* Freezing—Heat.
- Soil tubes, r., r. to p 40 (text and fn)
- Soils:
  - analyses 175, 198, 199
  - of New Jersey 77, 78
  - research, recommendations 59, 158, 159, 168, 169
  - *see also* names of kinds of soil—*also* headings beginning Earth, Soil—*also* Admixtures—Freezing—Heat—Lime content—Organic content—Subgrades—Subsoil.
- Sourwine, J. A.:
  - frost occurrence data, analysis method, a 51–60
- Spangles 177, 178
- Specific gravity:
  - of oils as affecting resistance to action of moisture 193
  - test results for various materials 70, 73, 192, 194, 210
- Specific heat, *see* Heat radiation.
- Specific viscosity, *see* Viscosity.
- Specifications, *see* American Society for Testing Materials—Brick, paving—Concrete aggregate—Concrete pavements (construction)—Culverts—Oils.
- Spelter, *see* Culverts.
- Spreading methods and equipment 189, 191, 200–208, 226, 258, 259
- Stability:
  - of bituminous road surfaces, factors affecting 129, 157, 158, 168, 190, 192–208
  - of subgrades, *see* Admixtures—Granular layers—Subgrades (construction)—Uniformity.
- Stability tests of bituminous paving mixtures 193, 194, 206
- Stage construction:
  - in undeveloped regions 159, 160, 168
  - *see also* Federal-aid.
- Stain tests, stain test formulas 195, 203
- Standards, American Society for Testing Materials 235, 236
- Stanton, T. E., formula 195, 196
- Static load-deflection data for various tire types 83, 85, 102, 103, 106–109, 131–136, 140, 144, 146, 151, 152
- St. Clair, G. P. (collab.):
  - calibrations of accelerometers, a 81–109, 111
- Steam as heating agent for road oils 201
- Steam vehicles, taxation, r 49
- Steel, *see* Concrete reinforcement—Culverts—Form installation—Steel-tired traffic.
- Steel-tired traffic:
  - highway design for 154, 155, 167
- Stiffness of concrete slabs, formula 2, 4
- Stone, stone roads:
  - impact test data, stone road 142, 144, 145, 150, 151
  - *see also* names of kinds of stone—*also* Aggregate, mineral—Base courses—Bituminous treatment—Block pavements—Concrete aggregate—Cover material—Crushed stone—Macadam pavements—Traffic-bound roads.
- Stop-watch studies of p[illegible] 247–260
- Stop watches, use, recomme[illegible] 254, 260
- Storage, *see* Materials, highw[illegible]
- Strain in concrete slabs:
  - relation to load, research 172
- Street car traffic, recommendations 167, 171, 172
- Streets, city:
  - design 166, 167, 171, 172
  - *see also* Classification—Finance—Improvement—Planning.
- Strength, *see* Age-strength—Compressive strength—Flexural strength—Pressure—Rupture—Tensile strength—Transverse strength.
- Stresses:
  - impact stresses—
    - reduction by nonrigid road surfaces 151
    - *see also* Impact.
  - in concrete bridge slabs—
    - wheel load stresses, computation, a 1–23
  - in concrete pavement slabs—
    - as affected by curing method 217, 220, 232, 234
    - compressive stresses, control by joints 185
    - r. to p 2 (fn), 8 (fn), 185
    - research 172
    - *see also* Cracks.
  - in steel reinforcement of concrete pavements 185
  - *see also* Freezing—Pressure.
- Subgrades:
  - construction methods, road congress data 154–156, 167, 184, 185, 187
  - subgrade condition as affecting—
    - oil-mixed road surfaces 193, 197, 198
    - time losses in concrete paving jobs 247, 248
  - *see also* headings beginning Granular, Soil—*also* Drainage—Freezing—Friction—Soils—Tar paper.
- Subsidies, *see* Grants.
- Subsoils:
  - frozen subsoils, Siberia, r. to p 79
  - research, recommendations 158, 168
- Subways 167, 171, 172
- Supercooling:
  - of air, of soils, of water 51, 52, 61–65, 115, 126
  - *see also* Freezing.
- Superelevation of bituminous pavements 168
- Superhighways (Holland) 162
- Supervision and inspection, importance 154, 155, 164, 167–170, 184, 186, 187, 246
  - *see also* Production studies.
- Surface hardness, surface wear, *see* Wear.
- Surface loss of untreated gravel and crushed stone roads (New Mexico) 197
- Surface-mixed roads, *see* Mixed-in-place.
- Surface treatment, *see* Bituminous treatment.
- Surtaxes, special taxes, motor vehicle 42, 45
- Surveys, *see* Culverts—North Carolina—Reconnaissance surveys—Tennessee—Traffic studies.
- Swampy areas, freezing of soils in 116
- Sweden, highways and highway finance 159, 163
- Sweeping, *see* Brooming methods.
- Swelling:
  - of aggregate as test for suitability for oil mixing 192
  - *see also* Expansion.

T

- Taber, Stephen:
  - freezing and thawing of soils as factors in destruction of road pavements, a 113–132
  - r., r. to p 62, 63, 79, 113 (fn), 117 (fn), 127 (fn)
- Tamping, *see* Compaction—Finishing.
- Tar:
  - types and use 69–71, 80, 157, 158, 168, 201–208
  - *see also* Bituminous materials—Cut-back products—Fillers—Paint coats—Priming coats—Seal coats—Tar paper—Tarvia.
- Tar paper on subgrade as concrete pavement curing agent 209–211, 213–220, 222–231
- Tarvia as concrete pavement curing agent 209–211, 213, 214, 216–220, 226, 227, 229–234
- "Tax-anticipation warrants," North Carolina 238
- Taxes as source of highway revenue:
  - excise taxes (in France) 161
  - fuel taxes, gasoline taxes 36, 39, 42, 43, 46, 159–164, 170; rear cover of No. 9
  - general taxes, property taxes, taxation of groups benefiting by highway improvements 158, 161–164, 169–171
  - motor vehicle taxes, user taxes—
    - apportioning, technical basis for, various tax bases considered, a 41–50
    - road congress data 159–166, 169–171
    - *see also* Registration fees—Taxes . . . (fuel—tire)
  - oil taxes 159, 163
  - privilege taxes (in Italy) 162
  - tire taxes 41–44, 46, 49, 162, 163
  - *see also* Assessments—North Carolina—Poll taxes—Revenues—Tolls—Traffic studies.
- Taylor, C. A. (collab.):
  - soil sampling with compressed air unit, a 40
- Temperature:
  - air temperature—
    - as affecting curing of quick-hardening concrete 188
    - relation to soil temperature, freezing studies data 51–64, 118, 120, 123–125, 128, 130, 131

Page

- Temperature—Continued.
  - application temperatures of bituminous materials, r. 157, 201–203, 206
  - of concrete; relative temperatures, uncoated, and bituminous coated concrete slabs 227
  - of ground below surface of freezing soil as affecting heaving 125
  - testing temperatures for various bituminous materials 70, 73, 192, 194
  - *see also* Asphalt concrete paving work—Bituminous paving mixtures—Climatic conditions—Cracks (in concrete pavements)—Fluxes (fluxing media)—Freezing—Heat—Supercooling—Temperature changes.
- Temperature changes as affecting:
  - concrete variously cured 211 214, 216, 218–228, 232–234
  - movement of concrete slabs, research 172
- Tennessee concrete curing methods study 209–234
- Tensile strength:
  - of cement (test results) 210
  - of concrete as affected by aggregate type 184 186
  - of soils (clay) 123, 126
- Tensile stresses, *see* Stresses—Tension.
- Tension:
  - in concrete pavements—
    - allowance for, in edge thickness calculation 185
  - in freezing substances, capillary tension in soils 63, 64, 68, 120, 122, 127, 128
  - *see also* Tensile strength.
- Test methods and apparatus, *see* names of tests, names of materials and structures tested, names of testing machines and apparatus.
- Thawing of soils 130
  - *see also* Freezing.
- Thickness:
  - of bituminous roads, bituminous binder courses, bituminous wearing courses 190, 196–208, 258
  - of concrete pavements, practice 154, 155, 167, 185
  - *see also* Base courses—Brick, paving—Edge thickening — Granular layers—Load capacities—Macadam pavements (in undeveloped countries)—Tires.
- Thompson, Prof. J. T.:
  - r., r. to p. .... 152 (text and fn)
- Thrusts exerted by expanding ice 62–64
  - *see also* Freezing.
- Time-displacement records, *see* Displacement.
- Time losses in paving jobs:
  - production studies data 247 260
- Time schedules, time-tables, in public road services:
  - recommendation 171
- Timing device in asphalt concrete paving work 259
- Tire-cutting machine 134, 139, 152
- Tires, motor vehicle:
  - tread thickness, tire type and condition as affecting truck wheel impact 81 87, 96–109, 111, 133–152
  - width and type as affecting road condition and costs 43, 46, 49, 50, 154, 155, 167
  - *see also* Steel-tired traffic—Taxes . . . (tire taxes).
- Togoland, road construction 160
- Tolls as source of highway revenue 162, 163
- Toms, R. E., road congress report 188
- Topeka type experimental pavement (Connecticut Avenue), data 69–76, 80
- Topsoil, *see* Sand-clay mixtures—Sand roads, sandy roads (sand-clay roads).
- Township roads, financing, *see* Finance—North Carolina.
- Traffic-bound roads:
  - construction method 203
  - *see also* Bituminous treatment.
- Traffic capacities:
  - of city streets, methods of increasing 166, 167, 171, 172
  - of roads of various types 153–158, 167, 168, 189, 199, 201 203, 205, 208
  - *see also* Load capacities.
- Traffic congestion:
  - as affected by use of motor trucks and busses 45, 46
  - remedies 166, 167, 171, 172
- Traffic islands 159, 171

Page

- Traffic lanes:
  - markings 171
  - width 159, 166, 169
- Traffic regulation 149, 153, 166, 167, 169, 171, 172
- Traffic signs and signals 153, 166, 171
- Traffic studies, traffic surveys:
  - in Michigan, announcement 172
  - value 171
  - *see also* Connecticut Avenue—North Carolina.
- Traffic volume, traffic intensity:
  - research, recommendations 158, 168
- Trailers, motor vehicle:
  - registrations, registration receipts (1929) 37, 38
  - taxation 49
- Transport, highway:
  - correlation with other means of transport, adaptation to collective and individual uses 153, 159, 163–167, 169–171
  - *see also* headings beginning Motor, Transportation.
- Transportation companies and enterprises, highway:
  - financing 166, 171
  - quasi-legal coordination, compulsory coordination 165, 170, 171
- Transportation costs, highway:
  - as affected by highway improvements 163, 164, 169
  - research, recommendations 158, 168
- Transverse strength of concrete:
  - relation to compressive strength 186
  - *see also* Cantilever beam tests—Flexural strength—Rupture—Transverse testing machine.
- Transverse testing machine for concrete beams 210, 211
- Trap rock:
  - as aggregate in bituminous macadam pavements 206
  - *see also* Aggregate, mineral—Block pavements—Concrete aggregate—Connecticut Avenue.
- Trip mileages:
  - of motor trucks and of motor passenger cars compared 165, 166, 171
  - *see also* Haul lengths.
- Truckee River, experimental road along:
  - materials used .... 195 (fn)
- Trucks, motor, *see* Motor trucks—Tires.
- Trumbower, Henry R.:
  - road congress report 165–166
- Tunis, road construction 160
- Turning time, *see* Production studies.
- Turntable operation in concrete paving jobs 249, 252, 253
- Twisting moments in concrete bridge slabs 2–7, 12–19, 22, 23
- Two-course concrete pavements 153–155, 167
- Two-lane roads in undeveloped regions 159, 169

U

- Undeveloped regions:
  - highways and highway finance 158–160, 164, 166, 168, 169, 171
- Uniformity:
  - of concrete—
    - as affected by method of proportioning 186, 187
    - as affecting smoothness of concrete surfaces 187
  - of concrete aggregate 187
  - of subgrades, importance 129, 154, 155, 167, 184, 185
  - of texture in soils as affecting differential heaving 113
    - *see also* Freezing.
- University of Illinois, *see* Westergaard, H. M.
- University of Iowa, State, *see* Flood water.
- University of South Carolina, *see* Taber, Stephen.
- Unloading, *see* Loading operations.
- User taxes, *see* Taxes . . . (motor vehicle taxes).

V

- Velocity of water flowing over embankments 30–34
- Viscosity of bituminous materials 70, 73, 192, 193, 195, 200–202, 204, 226, 227
- Voids:
  - in bituminous road surfaces, bituminous paving mixtures 195, 197, 205–207
  - in concrete as basis of principles of design 186
  - in soils as affecting ice segregation 113–132
    - *see also* Freezing.

Page

- Volumes, volume change *see* Densities and volumes — Expansion — Freezing — Shrinkage—Soil science.

W

- Water, *see* headings beginning Moisture, Water—*also* Absorption—Briquet tests—Concrete mix—Concrete mixers—Curing—Drinking fountains—Expansion—Flood water—Freezing—Moisture — Pressure — Traffic-bound roads.
- Water-cement-ratio method of design of concrete mixes 184–186
- Water content:
  - of soils—
    - percentage, average soil 115
    - *see also* Freezing—Soil flow—Soil moisture—Water table.
  - *see also* Concrete mix—Moisture—Moisture content—Water-cement-ratio.
- Water curing, *see* Curing.
- Water-logging of oil-mixed road surfaces 197
- Water table, ground:
  - position as affecting subgrade conditions 68, 78, 79, 122, 123, 128–132
- Waterproof paper, *see* Sisalkraft paper—Tar paper.
- Waterproof qualities of oil-mixed road surfaces 197
- Waterproofing of road surfaces, *see* Bituminous treatment.
- Waterways, *see* Transport, highway (correlation).
- Wear:
  - of concrete pavements—
    - as affected by—
      - curing method 211, 216, 217, 229, 230
      - tire type 154
        - *see also* Impact.
    - prevention (of abrasion) by 2-course construction 154
  - of cushion tires 138, 148, 152
  - of low-cost road surfaces, *see* Surface loss.
  - of rubber block pavements 157
  - tests, *see* Concrete aggregate (tests)—Rattler test—Wear machine.
- Wear machine for testing concrete pavements 211, 229
- Wearing courses for pavements of various types 74, 167, 202, 205, 206
  - *see also* Bituminous treatment—Concrete pavements.
- Weather, *see* Climatic conditions—Freezing—Rainfall—Temperature.
- Weighing equipment 187, 259
  - *see also* Materials, highway (handling).
- Weight:
  - of motor vehicles—
    - as tax basis 41–50
    - relation to road design 45
  - *see also* Load—Weight corrections—Weight method.
- Weight corrections, concrete aggregate 187
- Weight method of proportioning concrete 155, 167, 185–187
- Weirs, flow of flood water over 30–34
- Westergaard, H. M.:
  - computation of stresses in bridge slabs due to wheel loads, a 1–23
  - r. to p. .... 1 (fn), 2 (fn), 185
- Western roads, bituminous treatment 189–203, 207, 208
- Wheel, wheel loads, *see* Impact—Load—Reaction wheel—Stresses.
- Width:
  - of bridges, roads, traffic lanes 159, 166, 169
  - of concrete slabs as affecting crack formation 185
  - *see also* Rights of way—Tires.
- Woolf, D. O.:
  - method for measurement of water for cement briquet test, a 182–183
- Workability of concrete:
  - test, need for 186

Y

- Yard layout for concrete paving work 250–252 260
- Yarnell, David L., and Floyd A. Nagler:
  - flow of water over railway and highway embankments, a 30–34

Z

- Zinc spelter coatings, *see* Culverts.

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